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Elevation and frequency of occurrence of floods in Squaw Creek basin in Ames, Iowa

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ELEVATION AND FREQUENCY OF OCCURRENCE OF FLOODS IN SQUAW CREEK BASIN IN AMES, IOWA

by

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Signatures have been redacted for privacy

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INTRODUCTION

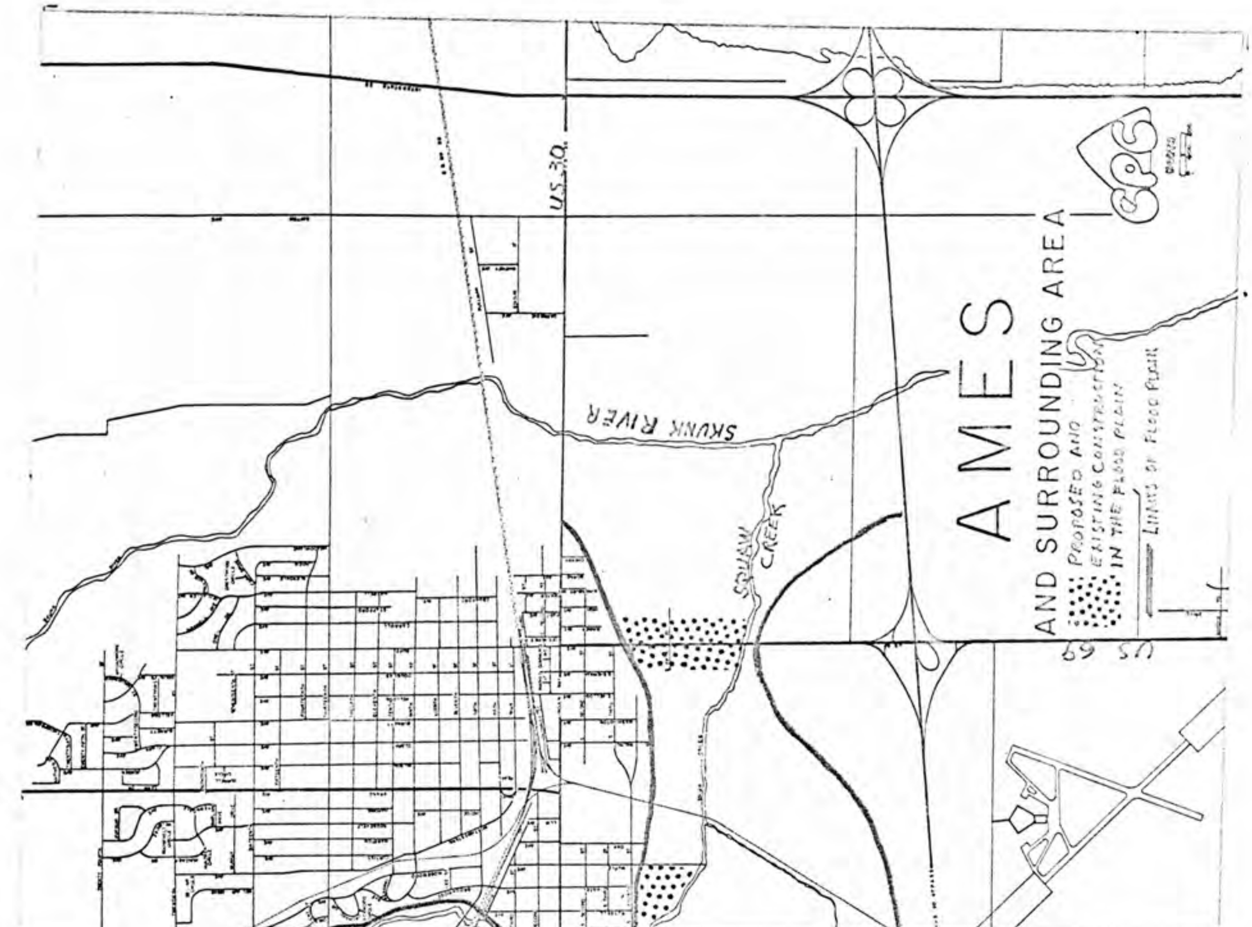
The entire Skunk River Basin in Iowa is unique in that no major urban damages have been experienced during floods. With minor exceptions, cities and towns are so located that they escape flood damages from streams. The vast majority of flood damages occurring in the basin are sustained by crops, although property losses, including damage to railraods, highways, bridges, utilities, and farm improvements, are of considerable magnitude (1).

The City of Ames, located at the junction of the Skunk River and its tributary, Squaw Creek, is mostly located on high ground. It has thus avoided major damages from the flooding of either stream. However, Wells has shown that the maximum flood potential of neither Skunk River nor Squaw Creek has been realized or even approached to date (2).

The flood plain of the reach of Squaw Creek within the city limits of Ames is generally used for athletic fields, public parks, pasture and crop land. In recent years, however, many commercial establishments have been located on the flood plain along South Duff Avenue; residences have been constructed on the flood plain between Squaw Creek and South Fourth Street; a multi-million dollar complex of University buildings has been proposed for location in the flood plain just south of Lincoln Way; and it has been proposed that the University Golf Course club house be relocated onto the flood plain just west of Stange road. Figure 1 shows the areas of completed and proposed construction in the Squaw Creek flood plain.

In view of the potential flood damage to buildings constructed in the Squaw Creek basin, the purpose of this thesis is to determine the elevation and frequency of occurrence of floods in the Squaw Creek basin in the city of Ames. The results of this study will be presented in such a form that they may serve as a basis for a reasonable determination of the flood hazard to any existing or proposed structures located in the flood plain. For each of several probable floods, a profile of the water surface will be prepared along Squaw Creek for its entire flow path inside the city of Ames.

Figure 1. Vicinity map -- Ames, Iowa


AMES
AND SURROUNDING AREA
PROPOSED AND EXISTING CONSTRUCTION IN THE FLOOD PLAIN
LIMITS OF FLOOD PLAIN
SKUNK RIVER
SQUAW CREEK
U.S. 30
US 69

HISTORICAL FLOODS

Official stream flow records for Squaw Creek are available only for the years 1919-1927. An official stream gaging station of the U. S. Geological Survey was maintained in Brookside Park from 1919 until 1925 when it was moved to the Lincoln Way bridge (3). The gaging station was discontinued in 1927. Unfortunately for the purpose of predicting future floods, no major floods occurred in the eight years of official record.

The following observations of historical flooding along Squaw Creek are summaries of accounts published in the Ames Daily Tribune and other references as noted. All statements enclosed by quotation marks appeared in the original newspaper article.

4 June 1918 This flood is apparently the flood of record with a peak discharge of 6,900 cfs as estimated by the U. S. Geological Survey (3). A major damage was the failure of the Lincoln Way Highway bridge (4). The two-lane bridge constructed as a replacement remained until 1958 when the present four-lane structure was erected. The two-lane structure was the site of the U. S. Geological Survey gaging station prior to discontinuance.

19 May 1944 "Squaw Creek, the highest it has risen in 20 years, has reached up to within a few yards of Beech Avenue." The University Golf Course was flooded as were several "victory" gardens in the flood plain.

13 June 1947 "At noon today, six families had been evacuated from their homes, three from the Skunk River area east of the city, and three from the Squaw Creek area at the end of South Maple, according to police reports." Flooding of the state-owned lands on both sides of Lincoln Way

was reported.

1 June 1954 "Squaw Creek flooded over South Riverside Drive, south of the bridge, this morning about 5 o'clock, and the road was closed to traffic at 5:30. Water still covered several acres of the flats south of Lincoln Way between downtown and the college."

28 August 1954 "Police moved five families out of the South Maple area Friday night. ...Water was up in much of the low areas, but not as high as it has been many other times. ... City manager John M.Carpenter directed city crews working at the South Riverside Drive bridge Friday night. He said the crest hovered within inches of the top of the bridge beams for over two hours." The flats between downtown Ames and the University were again flooded, as was Brookside Park.

5 July 1958 "Residents on South Maple Avenue south of Fourth Street were marooned for a time Friday when Squaw Creek flood waters overflowed South Maple Avenue between their homes and South Fourth Street. ... The bridge over Squaw Creek at South Riverside Drive was closed early Friday morning. Water apparently washed out a part of the county road just south of the bridge. ...The flood apparently ruined an irrigation test being conducted by Iowa State College south of Lincoln Way. Cattle in the same area had to be moved to higher ground near Beech Avenue. ...The College Golf Course along Squaw Creek was flooded under about four feet of water most of Friday morning. Brookside Park was flooded twice this week."

30 March 1960 "In the meantime, water has spread over lowlands on both sides of Lincoln Way at the Skunk River crossing and Squaw Creek bridge. Water covers East and West roads south of Ames, and South Riverside Drive

leading to the airport is under several inches of water. ...Squaw Creek is reported out of its banks along the entire stream extending northeast about 15 miles. It rose earlier than Skunk River, and by noon was nearly 12 feet deep, having flooded Brookside Park and University property south of there, and private property along the stream to Skunk River."

Views of some of these floods taken from various locations along Squaw Creek are shown in Figures 2, 3, 4, 5, 6, and 7. These photographs show that although extensive areas have been flooded in the past, the flooding has generally been relatively shallow.

While no official measurements of flood flow in Squaw Creek were made after 1927, Dr. Harris F. Seidel, Director of Water and Waste Treatment, Ames, Iowa, made crest measurements between 1954 and 1960 at various locations along Squaw Creek. His measurements, listed in Table 1, have been a valuable reference in this study.

Table 1. Recent Squaw Creek flood crests
(From notes of Dr. Harris F. Seidel, Director of Water and Waste Water Treatment, Ames, Iowa.)

	Location of Measurement					
	15th St.	6th St.	Lincoln Way	S. Riverside Drive	S. Duff	U.S.G.S. Gage Skunk River (below Squaw)
Bench mark	Conc Deck	Hand Rail	Hand Rail	Hand Rail	Bridge Rail	
Elevations	909.46	914.48	906.56	898.76	897.65	867.10
DATES						
May 28/54	896.1		895.0	890.4	882.7	*6.2 (1:00 PM)
June 1/54						*10.86 (11:00 AM)
June 11/54				884.6	881.2 (10:00 AM)	*11.82 (8:00 AM, 11:00 AM)
Aug 28/54	898.1		893.6	891.7	884.1	*12.11 (3:00 AM)
June 1/57	897.6 (3:00 PM±)		893.4 (4:30 PM)	890.5 (3:30 PM)	882.7 (1:00 PM)	*11.49 (9:00 AM)
July 2/58	898.2	894.2	893.3 (4:30 PM)	890.6 (4:30 PM)	882.7 5:00 PM)	*11.12 (4:30 PM)
July 4/58	898.6 (6:00 AM±)	895.5 (8:00 AM±)	894.2 (8:00 AM)	891.5 (9:30 AM)	884.3 (11:30 AM)	*12.80 *1:30 PM)

Table 1. (Continued)

	Location of Measurement					
	13th St.	6th St.	Lincoln Way	S. Riverside Drive	S. Duff	U.S.G.S. Gage Skunk River (below Squaw)
DATES						
March 30/60	897.5 (10:00 AM±)		893.0 (1:00 PM±)		883.7 (6-9 PM)	*13.2 (10:00 PM)

*Gage heighth

NOTES: Elevations shown are U. S. Geological Survey datum, which is City datum plus 825.55 Feet. The U. S. Geological Survey gaging station is on Skunk River, about 1/4 mile below the junction of Squaw Creek and Skunk River; 10 foot stage is bank full at the gaging point; Above this elevation, general bottom flooding occurs.

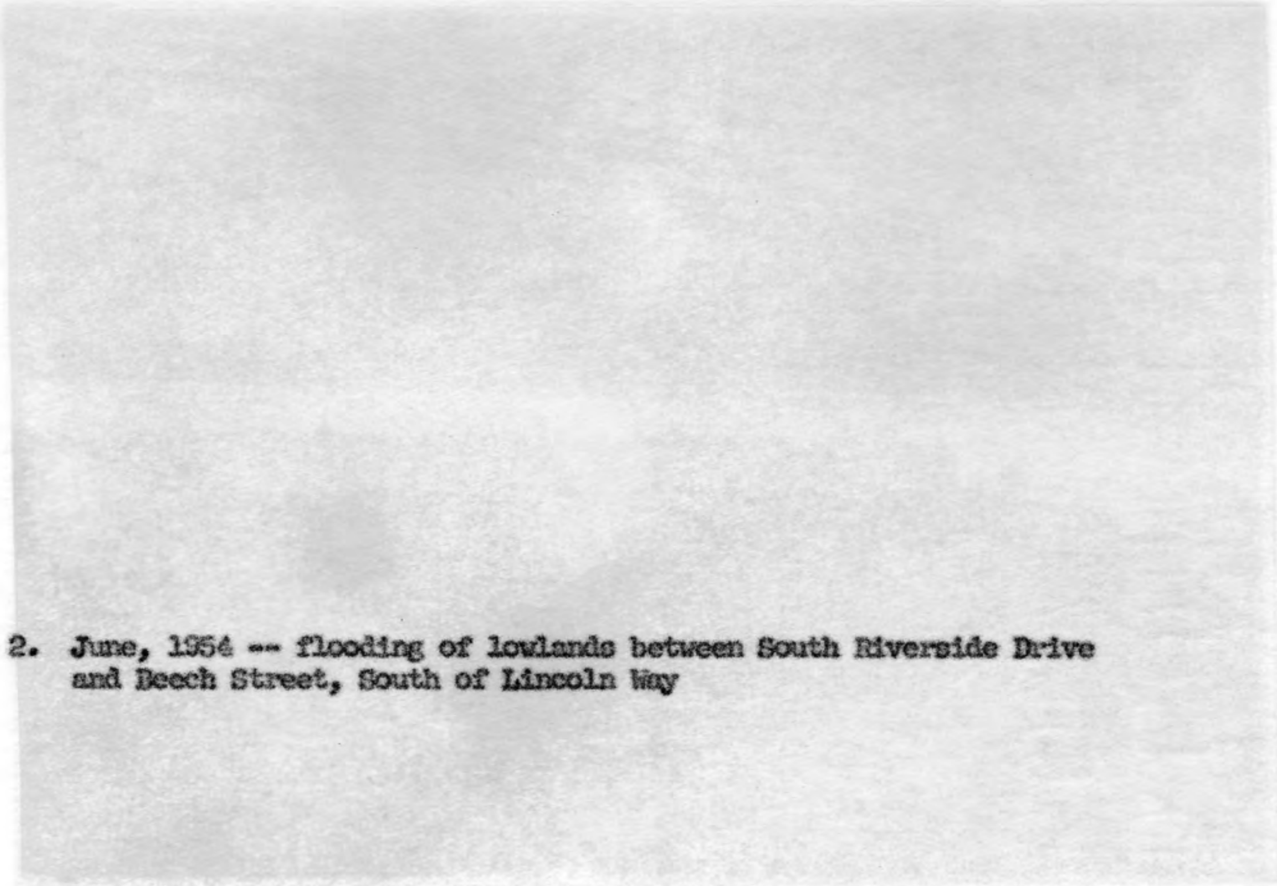
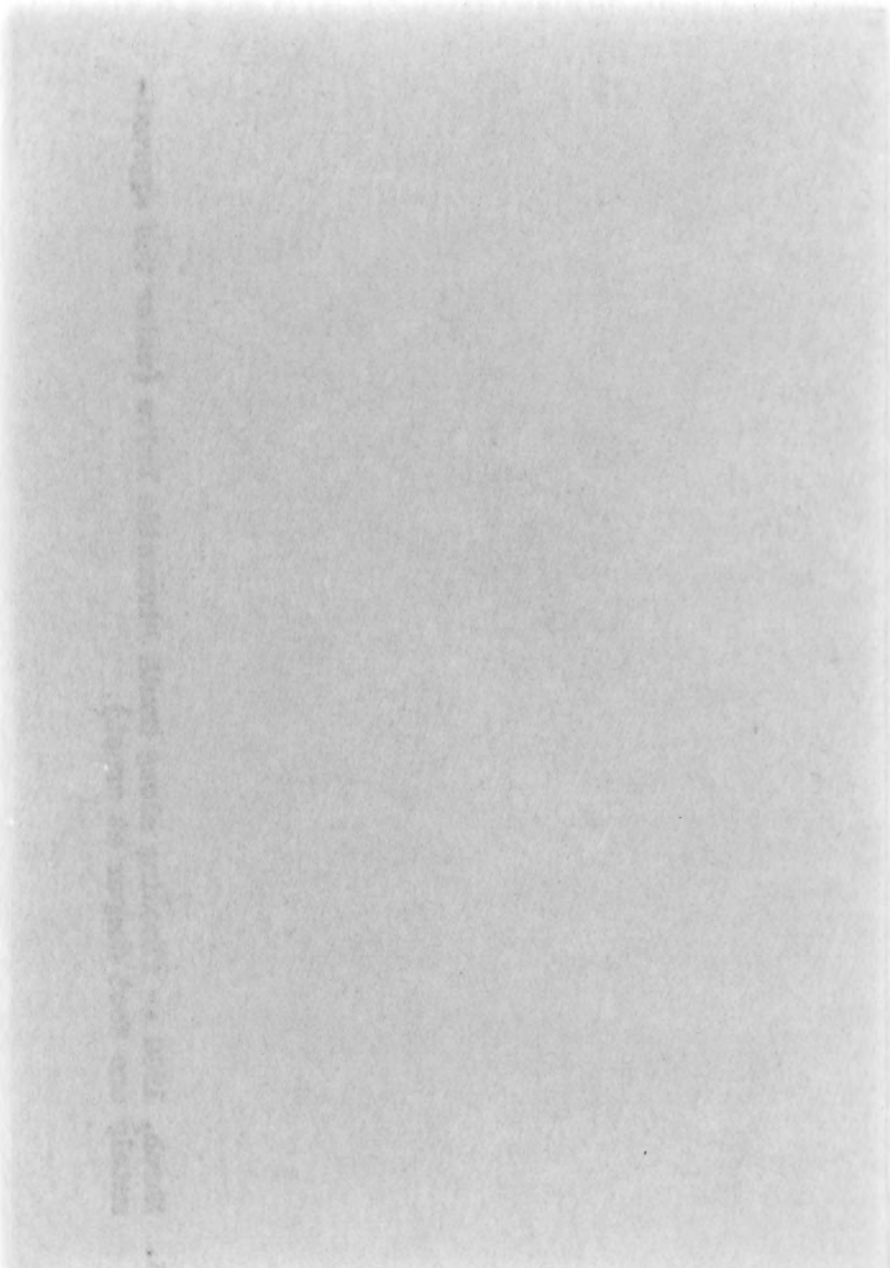


Figure 2. June, 1954 -- flooding of lowlands between South Riverside Drive and Beech Street, South of Lincoln Way



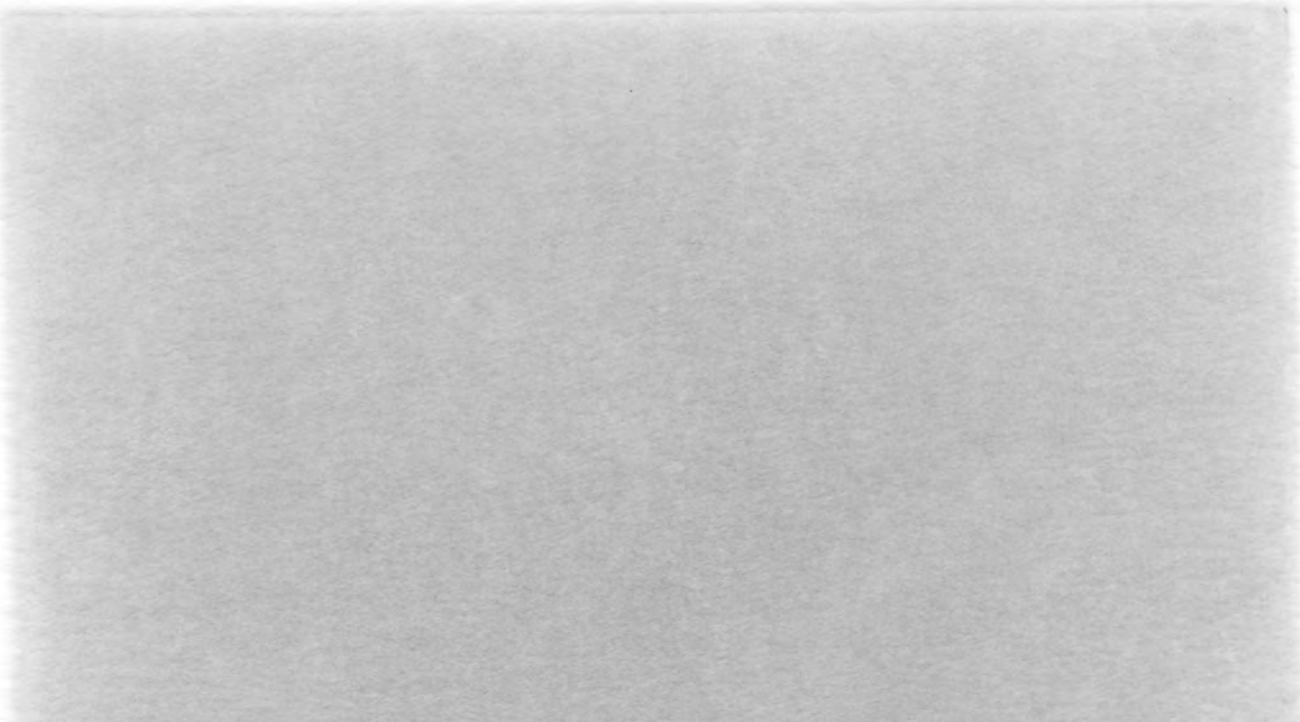
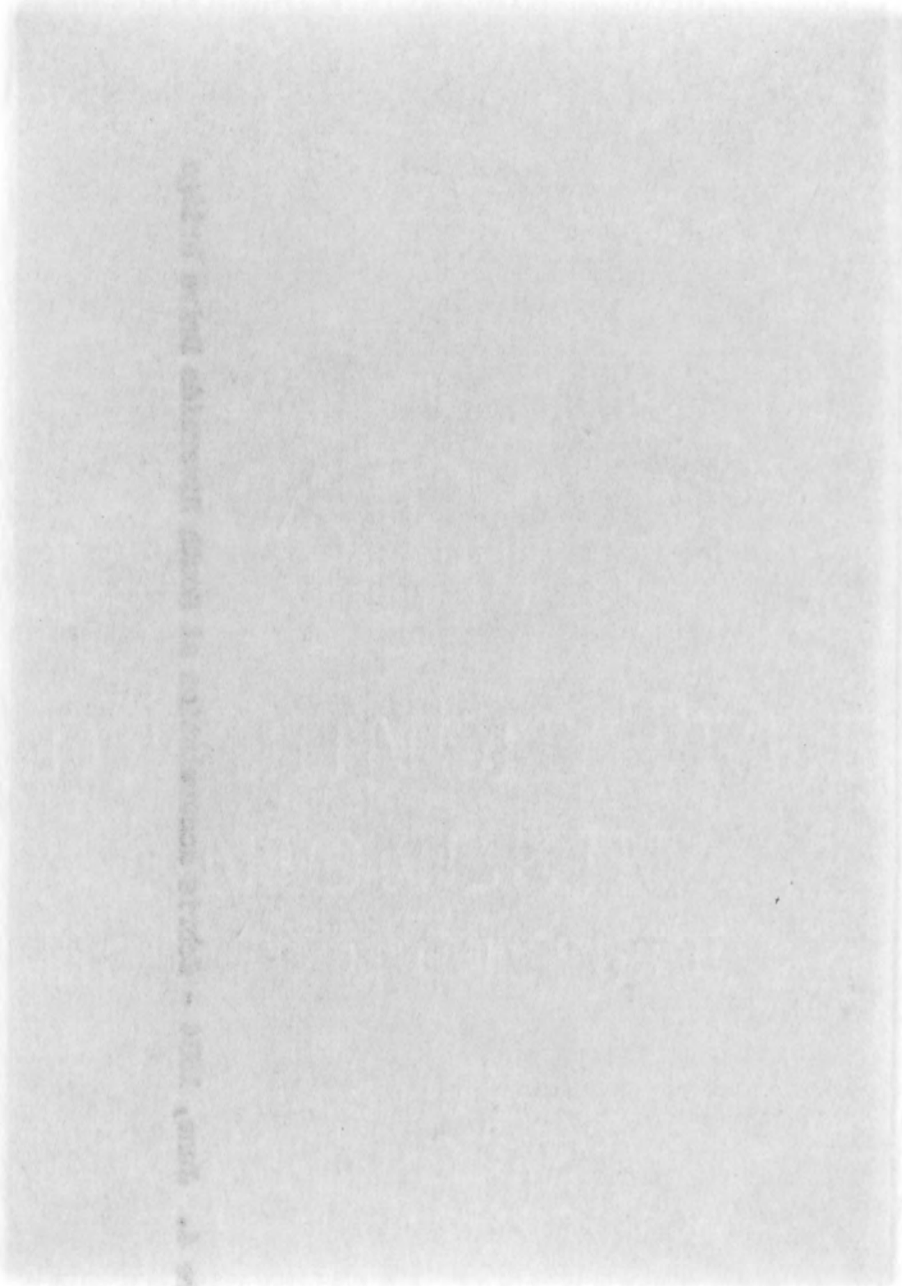


Figure 3. March, 1960 -- flooding along South Riverside Drive (water was approximately one foot deeper at crest)



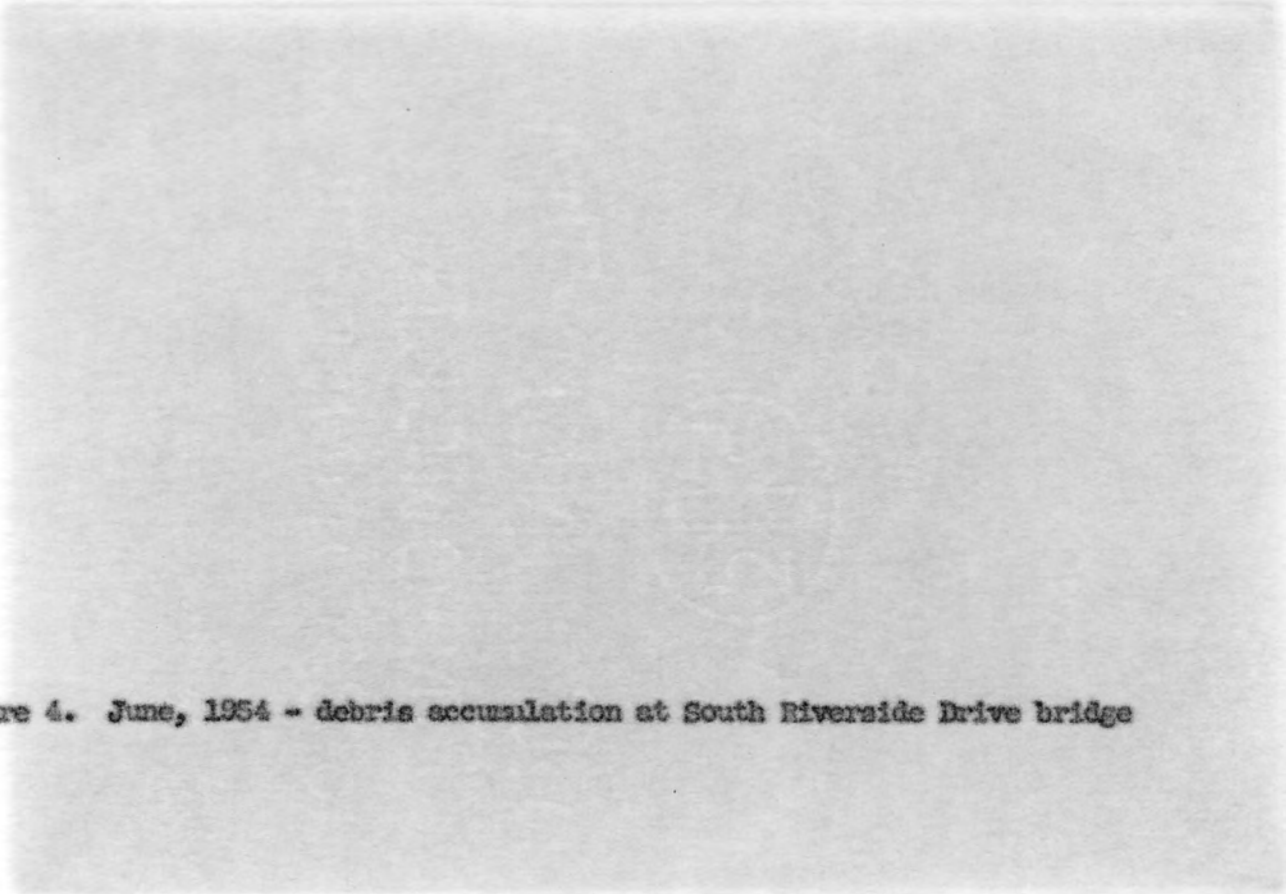


Figure 4. June, 1954 - debris accumulation at South Riverside Drive bridge

Figure 5. August, 1954 -- flooding at the foot bridge in Brookside Park

Figure 6. August, 1954 — flooding of the University Golf Course
(The area is being considered as the new location of the University Golf Course Club House.)

Figure 7. August, 1954 -- Flooding of the University Golf Course

Figure 7. August, 1954 -- flooding of the University Golf Course

SELECTION OF FLOOD FLOWS

General

Proper design of a structure to be erected in a flood plain requires consideration of certain information concerning the flood hazard. Of primary concern is the depth of flooding which can reasonably be expected to occur during the useful life of the structure. The basic hydraulic factors desirable for use in the design of most structures subject to flooding are frequency of occurrence, magnitude and water surface elevations of floodwaters.

At any given location along a stream the water surface elevation can be estimated for a given flow of floodwater, because since the stage-discharge relationship is a function of the physical features of the site. The frequency-discharge relationship is normally determined by statistical analysis of past records for the stream in question, or by regional correlation techniques.

The frequency-discharge relationship is normally expressed by terms such as the "ten year flood" or the "flood of 10 per cent chance of annual occurrence". Both terms have been used to describe the same flood; however, the percent-chance-of-annual-occurrence terminology is less misleading and will be used in this study. The probability of a 10 per cent chance of annual occurrence flood in any given year is one in ten. Similarly, the probability of a 2 per cent chance of annual occurrence flood in any given year is two in one hundred or one in fifty.

The above terminology is applicable to floods of relatively low magnitude which occur periodically. Other floods of great magnitude which are too rare to be assigned a frequency may also occur. These floods range from the maximum experienced in other watersheds in the region of interest to the probable maximum flood as computed from meteorological data.

Flood Flows Desired

It was planned that floods of 10, 4, 2, and 1 per cent chance of annual occurrence would be studied in order to provide the necessary correlation between water surface elevation and probability of occurrence.

Two additional flood flows were selected as representative of the high flows which could reasonably be expected to occur in this region, but for which frequencies could not be assigned. The flood flows chosen were the estimated discharges resulting from transposition of a great storm which occurred in northern Wisconsin on 28-31 August 1941. The additional two flows selected result from different assumptions of watershed conditions during the transposed storm. The first assumed condition is that the physical conditions now existing in the watershed remain unchanged. The second assumed condition is that the Gilbert Flood Control Reservoir proposed for construction by the Corps of Engineers, U. S. Army, in 1951 has been completed prior to the arrival of the storm. The second assumed condition includes the stipulation that the water surface in the flood control reservoir is at spillway crest elevation and that the outlet works are closed and inoperative when the storm occurs.

Flood Flows Selected

The transposed storm, designated UMV 1-22 by the Weather Bureau, was studied both by Wells (2) and the Corps of Engineers (5). Wells calculated the peak discharge to be expected at the mouth of Squaw Creek from the storm, while the Corps of Engineers limited their study to the discharge anticipated above the proposed Gilbert Reservoir. The discharge in Squaw Creek computed by Wells was chosen for use in this study together with Wells' simultaneous flow in Skunk River. Wells found the flow, unattenuated by the reservoir, to be 37,300 cfs in Squaw Creek and 79,400 cfs in Skunk River below the mouth of Squaw Creek. Wells made no study of reservoir attenuation of the expected run-off.

The Corps of Engineers designated storm UMV 1-22 as the Standard Project Storm for the Gilbert Reservoir and designated the resulting flood the Standard Project Flood. A Standard Project Storm may be defined as the greatest storm which can reasonably be expected to occur over a given basin. Although it would be an extremely rare event, it is of a lesser magnitude than the probable maximum storm which might occur. The Corps of Engineers computed discharge from the reservoir under conditions stipulated previously was 18,600 cfs (5). An arbitrary flow of 8,000 cfs was chosen as the Skunk River contribution to flooding at its junction with Squaw Creek. A flow of 8,000 cfs is approximately bankfull flow on the Skunk River. Therefore, the flows selected for this second condition were 18,600 cfs in Squaw Creek, and 26,600 cfs in Skunk River below the mouth of Squaw Creek. (See page 31).

The determination of flood flows corresponding to the desired specific frequencies posed a difficult problem. Normal methods of analysis require peak flood flow data for a continuous period of years exceeding the desired return interval. For example, it is desirable to have approximately 100 years of record when predicting flows of 2 per cent chance of annual occurrence. Such data were not available for Squaw Creek.

A 1953 Iowa Highway Research Board publication provided data from which flood flows of given frequency could be estimated for gaged and ungaged streams in Iowa (6). In this publication, the State of Iowa was divided into eight geographical regions. For each of the regions, a curve was developed which showed the relationship between stream watershed area and the basin mean annual flood. With the mean annual flood known, floods of recurrence interval up to 50 years could be estimated. Unfortunately, for the region encompassing Squaw Creek the appropriate curves were valid only for watershed areas exceeding 300 square miles, whereas Squaw Creek drains only 227 square miles.

A study using similar methods was made by the Iowa Natural Resources Council in 1962 (7). This study also had the limitation of providing data for basin areas in excess of 300 square miles in the Squaw Creek region. However, this study made use of an additional eleven years of basic data and provided a means of estimating floods of return intervals up to 100 years. The Iowa Natural Resources Council data were chosen as a basis for further estimation.

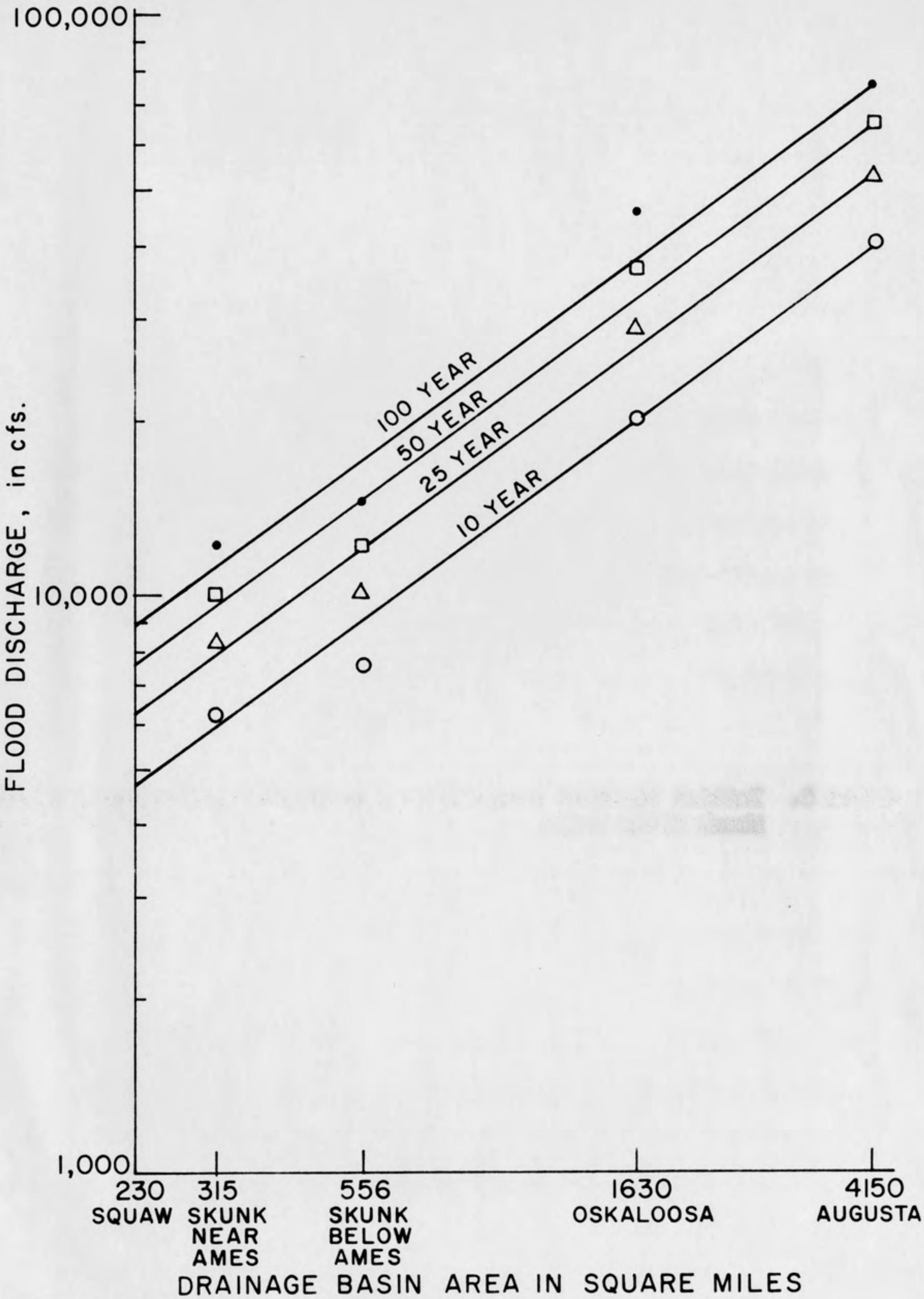
Data were obtained from the Iowa Natural Resources Council report from which curves were constructed showing the relationship between the size of drainage area and peak flood flows of the desired frequency for streams in the region encompassing Squaw Creek. The specific points from which the curves were plotted were for the U. S. Geological Survey stream gaging stations on Skunk River at Augusta, Oskaloosa, near Ames below Squaw Creek, and near Ames above Squaw Creek. The curves were extrapolated toward the smaller drainage areas, and the desired Squaw Creek flood flows were determined as illustrated in Figure 8.

The flows initially determined by this method and the corresponding Skunk River flows are as follows:

Per cent chance of Annual occurrence	Flood flow (cfs)	
	Squaw Creek (at Ames)	Skunk River (below Squaw Creek)
10	4,700	7,500
4	6,200	10,000
2	7,600	12,200
1	8,900	14,600

The Iowa Natural Resources Council study of flood frequencies at these four stations included a mathematical correction for short term records which existed at stations at Oskaloosa and near Ames below Squaw Creek. This correlation correction was introduced to provide an estimate of the long term record which could be expected at each of the two stations. The correction was computed by a statistical correlation technique of comparing records from stations of short terms of record with those of long terms of record in areas which are reasonably hydrologically homogeneous. The lengths of record which existed at each station are:

Figure 8. Initial regional analysis of area-discharge-frequency relations, Skunk River basin



100,000
10,000
1,000
FLOOD DISCHARGE, in cfs.
100 YEAR
50 YEAR
25 YEAR
10 YEAR
230
SQUAW
315
SKUNK
NEAR
AMES
556
SKUNK
BELOW
AMES
1630
OSKALOOSA
4150
AUGUSTA
DRAINAGE BASIN AREA IN SQUARE MILES

Recording Stations	Length of Stream Flow Record	Total Years
Near Ames, above Squaw Creek	1920-27, 1932-62	39
Near Ames, below Squaw Creek	1952 - 1962	11
Oskaloosa	1945 - 1962	18
Augusta	1914 - 1962	49
*Base period for correlation *1918	- 1962	45

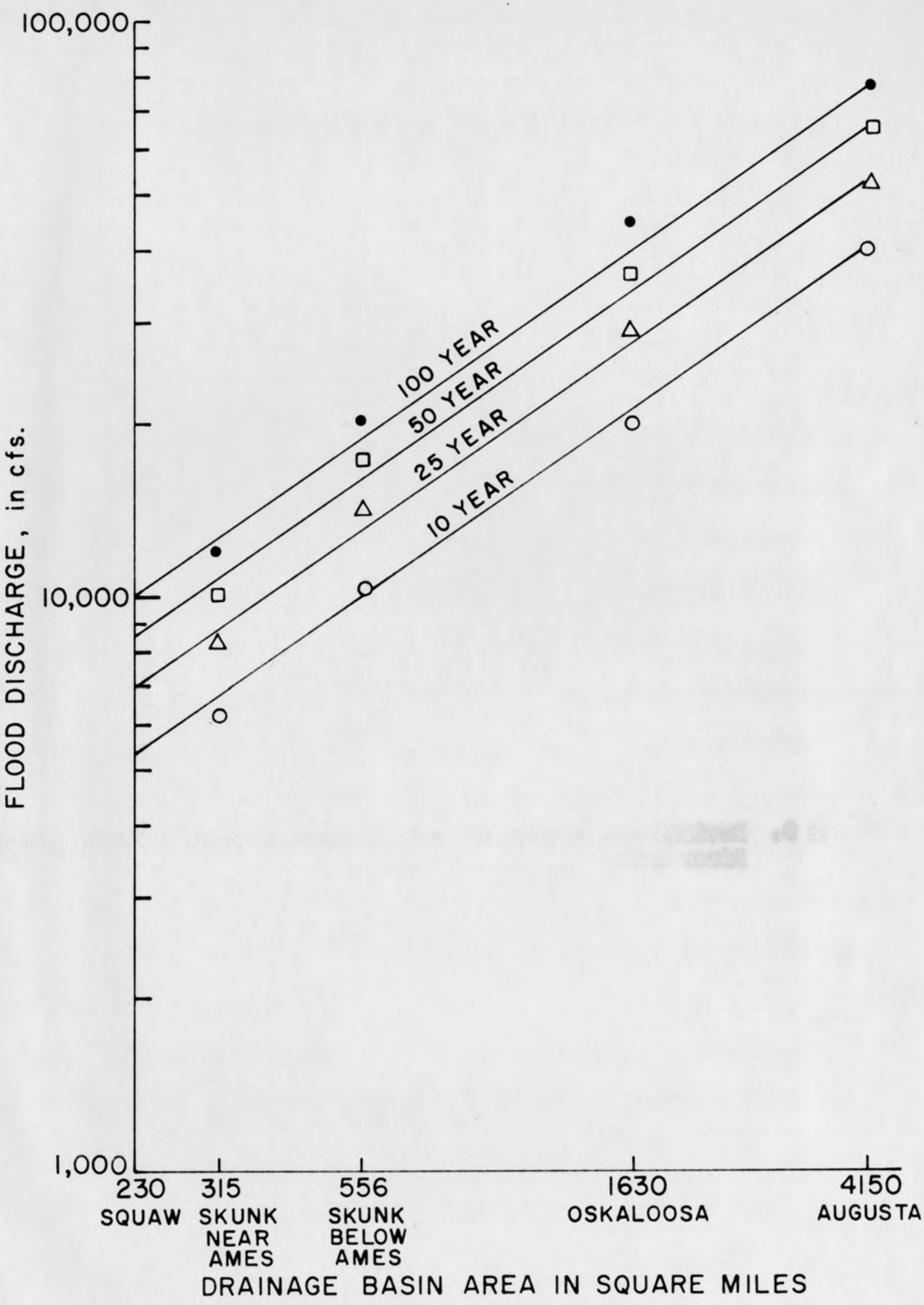
As water surface profiles were being computed using the above flow values, it was noted that computed water surface elevations were not in accordance with observed facts. The particular discrepancy noted was that the water surface calculated for the 10 per cent chance of annual occurrence flood flow did not overtop South Riverside Drive. Since overtopping had been experienced several times since 1944, all data, calculations and techniques were re-examined in an effort to determine if a discrepancy existed.

The re-examination disclosed that the correlation correction for the Skunk River below Squaw Creek, although correct in magnitude, had been misapplied, having been subtracted rather than added.

The data were corrected and new curves showing the drainage area - discharge - frequency relationship were constructed as shown in Figure 9. It is to be noted from a comparison of Figures 8 and 9 that the flood frequency values for the Skunk River below Squaw Creek are all higher in magnitude in Figure 9. This verifies the existence of and corrects the initial discrepancy noted.

From the new curves shown in Figure 9, the following flood flows for

Figure 9. Revised and adapted area-discharge-frequency relation, Skunk River basin


100,000
10,000
1,000
FLOOD DISCHARGE, in cfs.
100 YEAR
50 YEAR
25 YEAR
10 YEAR
230
SQUAW
315
SKUNK NEAR AMES
556
SKUNK BELOW AMES
1630
OSKALOOSA
4150
AUGUSTA
DRAINAGE BASIN AREA IN SQUARE MILES

Squaw Creek and Skunk River were obtained:

Per cent Chance of Annual Occurrence	Flow (cfs) Squaw Creek (at Ames)	Skunk River (below Squaw Creek)
10	5,400	10,500
4	7,000	14,200
2	8,600	17,100
1	10,000	20,500

The results of water surface profile computations obtained prior to the discovery of the error in the flood flows calculated for each per cent chance of annual occurrence remained valid if the true frequency occurrence could be determined for the flows used. In order to determine the true frequency, a curve was plotted on Gumbel (8) probability paper showing flood flows vs. recurrence interval in years, using the corrected data from Figure 9. From this curve, shown in Figure 10, the recurrence interval for the flows used in the calculations already completed could easily be determined. For example, the flow of 4,700 cfs which was initially assumed to have a recurrence interval of 10 years, is seen to have an actual return interval of 7.1 years, or a 14.1 per cent chance of annual occurrence.

Additional water surface profile calculations were made for the revised 10 per cent and 1 per cent chance of annual occurrence floods. The flows for which water surface profiles were finally computed are as follows:

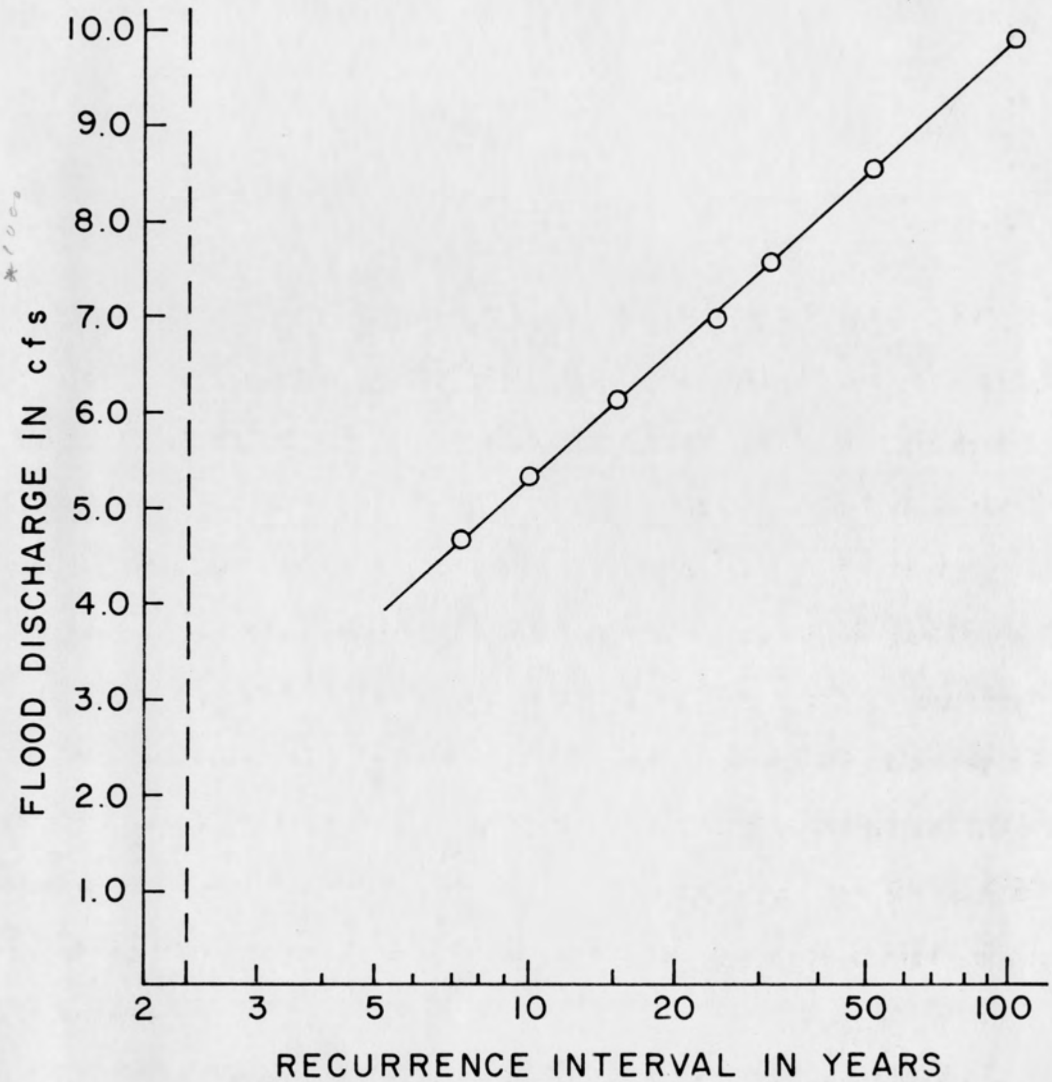


Figure 10. Adjusted return intervals for computed flood discharges

Per cent chance of Annual Occurrence for Squaw Creek	Peak Flood Flow for Squaw Creek cfs	Coincident flow Adopted for Skunk River (Below Squaw Creek) cfs
14.1	4,700	7,500
10.0	5,400	10,500
6.7	6,200	10,000
3.2	7,600	12,200
1.0	10,000	20,500
Standard Project Flood as discharged from Gilbert Reservoir	18,600	26,600
Standard Project Flood occurring under natural conditions	37,300	79,400

It should be noted that the magnitudes of flows in the Skunk River below Squaw Creek coincident with Squaw Creek flows of 14.1, 6.7, and 3.2 per cent chance of annual occurence are slightly lower than the desired values indicated in Figure 9 and in the table derived from Figure 9. It was initially reasoned that the maximum stage which reasonably could be expected below Squaw Creek, at the time of the Squaw Creek peaks, would be the coincident frequency and related flood discharge of the combined Skunk River and Squaw Creek flood flows. This is believed to be a more reasonable estimate of coincident conditions than would be the numerical addition of the respective flood flows of equal frequency. One would seldom expect the storm conditions in each basin to be such as to produce simultaneous frequency and magnitude of discharge.

Calculations subsequently were made which indicated that less than 0.05 foot error was introduced in water surface elevation calculations at South Duff Avenue by using the reduced Skunk River flow values, and no error was introduced upstream from that point.

COMPILATION OF BASIC DATA FOR COMPUTATIONS

Topographic Data

Information pertaining to stream drainage areas was obtained from data published by the Iowa Highway Research Board (9). Topographic maps of the Squaw Creek flood plain within the City of Ames were procurred from the Office of the City Engineer, Ames, Iowa. These maps were drawn to a 1 inch to 100 foot scale with a 2 foot contour interval. The maps were compiled from aerial photographs taken 5 May 1954, by the Abrams Aerial Survey Corporation, Lansing, Michigan.

Survey maps of the Skunk River channel between the junction of Squaw Creek and Skunk River and the U. S. Highway No. 30 By-pass bridge were obtained from the Iowa State Highway Commission. These maps were prepared by the Iowa State Highway Commission for planning purposes during the design of the U. S. Highway No. 30 By-pass bridge, and have no particular identification code. However, they are on file in the Hydraulic Design Section, Iowa State Highway Commission. These maps were prepared to a 1 inch to 100 foot scale and showed point elevations along the river channel and portions of the flood plain.

Iowa State Highway Commission plans were obtained for the widening of South Duff Avenue which is to be accomplished during the summer of 1963. The proposed road-grade elevations were used in water surface profile computations in which the flood flows caused flow over the road.

All elevations used were city datum, which is U. S. Geological Survey datum of 1929 minus 825.55 feet.

Bridge Data

Scale drawings of all bridges were made from field measurements. The drawings were the source of data used in bridge conveyance and backwater calculations. The upstream bridge and approach to the bridge were photographed from points near the center of the channel. The photographs were a useful supplement to notes taken in the field.

Layout plans of the bridges located at South Duff Avenue, Lincoln Way, Sixth Street and Thirteenth Street were obtained from the Iowa State Highway Commission. These plans were used only to verify bridge measurement data. Only field notes of channel widths and depths through the bridges were used in this study, however, as variation between planned and existing dimensions were noted.

Stream and Flood Plain Characteristics

The physical measurements of the stream channel and flood plain were taken from the topographic maps purchased from the office of the City Engineer, Ames, Iowa. The stream width and depth measurements from the maps were checked by field measurements and found to be quite accurate.

The land use data were taken from the topographic maps and verified by field observations.

Manning's roughness coefficient values for the flood plain were determined by comparing field analysis with written descriptions of areas of known "n" values described by Chow (10).

The roughness coefficient for the stream channel was determined by comparison of written descriptions of channels and suggested coefficient values from numerous sources, and by comparison of photographs taken of Squaw Creek with photographs of streams with known roughness values compiled by Chow.

Values used in this study to represent summer conditions were:

Location and Description	Roughness Coefficient
Stream Channel	0.040
Flood Plain	
Cultivated Areas	0.060
Pasture	0.040
Open Timber - such as Brookside Park	0.10
Heavy Timber and Scrub Undergrowth	0.12
Scrub Undergrowth only	0.11

In order to check the validity of the roughness coefficient chosen for the stream, a stage-discharge curve was reconstructed for the old U. S. Geological Survey gaging station located in Brookside Park, using information published by the U. S. Geological Survey. A cross section of the stream was constructed from the topographic maps. Using the stream slope as measured from the topographic maps, the cross sectional area and wetted perimeter as measured on the plotted cross section, and with a discharge taken from the stage-discharge curve, the value of the corresponding roughness coefficient was computed from Manning's equation in the form:

$$n = \frac{1.486}{Q} \quad AR^{2/3} S^{1/2} \text{ - - - - - - - - - - - - - - } (1)$$

where:

n = Roughness coefficient.

Q = Discharge in cfs.

A = Area of water cross section normal to direction of flow at a given water surface elevation (Sq. Ft.).

R = Hydraulic radius at the given water surface elevation (Ft.).

S = Stream surface slope in Ft./Ft.

Values obtained for a six foot variation in flow elevation ranged from 0.035 to 0.040, which verified the values previously selected. This technique of verifying the roughness values is believed to be substantiated by analysis of field conditions. Evidence indicates that Squaw Creek in the area of interest has meandered considerably. In general this indicates that the stream is no longer down cutting, and the slope is constant. Since a meandering stream is generally side-cutting, and there is evidence that the west bank of the channel is sloughing in the area of interest, the cross sectional area of the stream is probably greater at a given water surface elevation than it was when the gaging station was established. The assumed increase in area and hydraulic radius would result in a slightly higher value of roughness. This was accepted as being conservative.

The derived roughness coefficient values are applicable for summer floods only. Somewhat lower values should be used in investigation spring or snow-melt floods.

CALCULATION OF WATER SURFACE ELEVATIONS

Water surface elevations in sections of the stream and flood plain unobstructed by bridges were calculated by the Leach method as described by King (11). To use the Leach method, the stream was first divided into relatively short reaches. The conveyance of the average section of each reach was then calculated for various water surface elevations, and a curve of water surface elevation vs. conveyance was constructed. The reaches used in this study are shown in Figure 11.

Conveyance is defined as the parenthetical portion of Manning's equation written in the form:

$$Q = \left[\frac{1.486}{n}\right] AR^{2/3} S^{1/2} \text{ - - - - - - - - - - - - - - } (2)$$

then:

$$K = \frac{1.486}{n} AR^{2/3} \text{ - - - - - - - - - - - - - - - - } (3)$$

where:

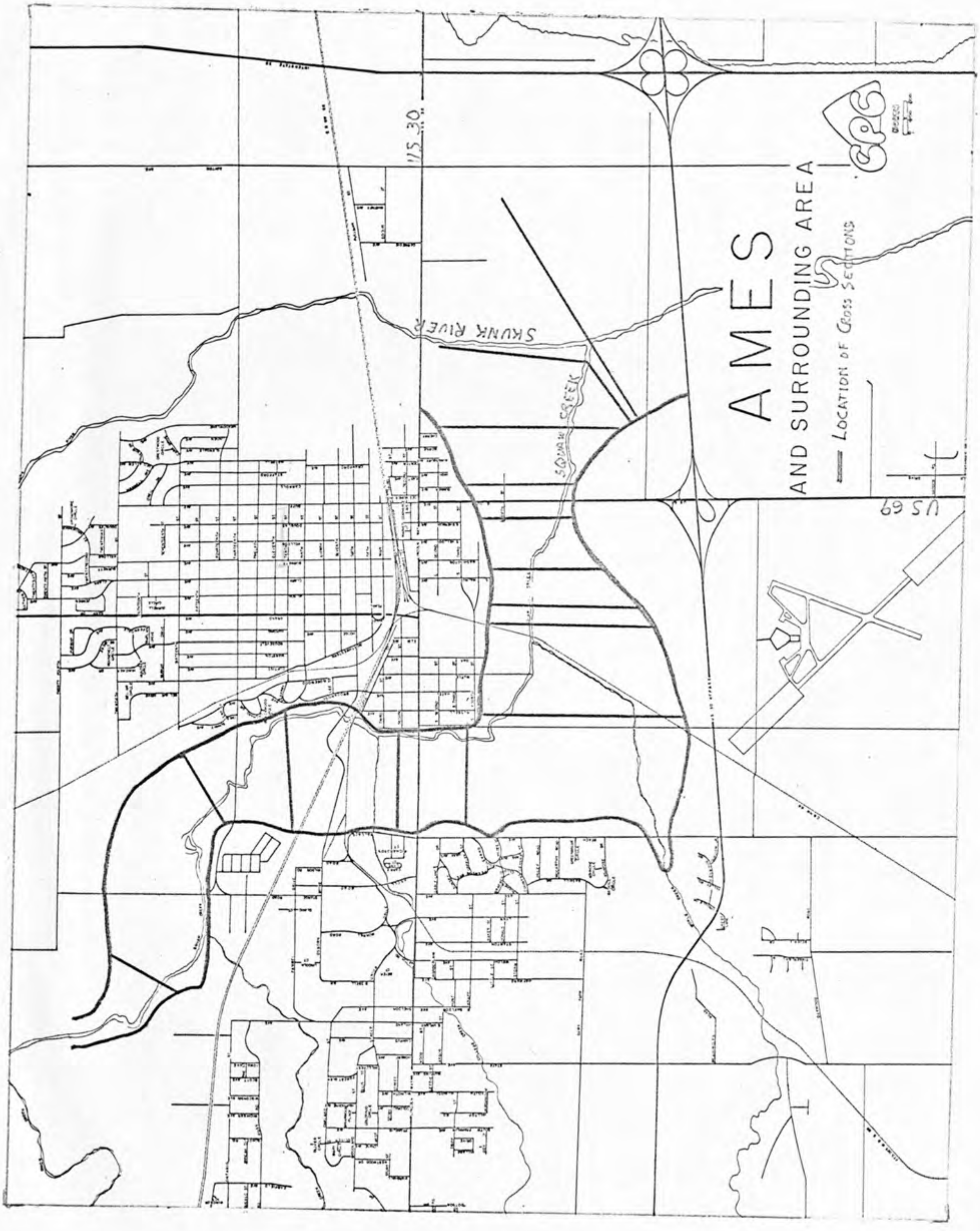
K = Conveyance at a given water surface elevation.

and other terms as defined for Equation 1.

For this study, the average or mean conveyance of a reach was calculated as the geometric mean of the conveyances of the end-sections of the reach, as suggested by Chow:

$$Km = \sqrt{KuKd} \text{ - - - - - - - - - - - - - - - - - - } (4)$$

Figure 11. Locations of valley cross sections


AMES
AND SURROUNDING AREA
LOCATION OF CROSS SECTIONS
SKUNK RIVER
SQUAW CREEK
US 30
US 69

where:

Km = Mean conveyance of the reach at a given water surface elevation.

Ku = Conveyance at the upstream end of the reach at the given elevation.

Kd = Conveyance at the downstream end of the reach at the given elevation.

Using values for Km calculated for several water surface elevation, a curve of water surface elevation vs. Km was constructed for each reach.

In order to use the Leach method, it was necessary that the water surface elevation at the lower end of the reach be known. The location of the U. S. Geological Survey gaging station on Skunk River below Squaw Creek was taken as the lower end of the first reach considered in this study.

To compute the water surface elevations in a reach, the water surface elevation at the mid-point was assumed for a given flow condition. Km was then determined from the Km vs. water surface elevation curve for the reach, and the average slope of the water surface in the reach calculated using Manning's equation in the form:

$$S = \frac{Q}{Km}^{2} \quad \text{- - - - - - - - - - - - - - - - - - - -} \quad (5)$$

where:

S = Average water surface slope in the reach.

Q = Flood flow for which the water surface elevation is desired (cfs).

Km = The mean conveyance of the reach at the assumed elevation.

The rise in water surface elevation between the lower end of the reach and the mid-point was then calculated by multiplying the calculated slope by one half the length of the reach:

$$R = \frac{SL}{2} \quad \text{- - - - - - - - - - - - - - - - - - -} \quad (6)$$

where:

R = Rise in water surface elevation between the lower end of the reach and the mid-point of the reach (Ft).

S = Average slope as calculated from Equation 5.

L = Length of the reach (Ft).

The calculated rise was then added to the known water surface elevation at the lower end of the reach in order to obtain the elevation of the water surface at the mid-point of the reach. In this study the calculated slope was assumed correct if the calculated water surface elevation at the mid-point was within 0.05 foot of the assumed elevation. If the assumed and calculated elevations did not agree within the desired limits of accuracy, a second elevation was assumed and the calculations repeated.

When a satisfactory value of the average slope had been calculated, the total rise of the water surface in the reach was determined by multiplying the slope by the length of the reach. The total rise was then added to the known elevation at the lower end of the reach to determine the elevation of the water surface at the upper end of the reach. This elevation then became the known elevation at the lower end of the next reach upstream. Water surface elevation calculations could then be made for that reach and continued upstream in a similar manner.

In this study, when a reach included a constriction caused by a bridge and its approach road or railroad embankment, the normal water surface elevation first was computed as if no constriction existed. This was accomplished by computing Kn for the reach as described previously. The water surface elevation vs. Kn curve constructed for the reach did not reflect the effects of the constriction. With the normal water surface elevation computed for the entire reach, the normal water surface elevation at the bridge crossing could be determined by interpolation between the end sections.

The normal water surface elevation at the bridge was then used in computation of the backwater caused by the constriction, using the method adopted by the U. S. Bureau of Public Roads (12). The expression used for calculating backwater caused by bridge constriction of the channel is as follows:

$$h^* = K^* \frac{V_n^2}{2g} + \alpha \left(\frac{A_n^2}{A_1^2} - \frac{A_n^2}{A_2^2} \right) \frac{V_n^2}{2g} \quad \text{- - - - - - - - - (7)}$$

where:

h^* = Total backwater above the computed normal water surface elevation (Ft).

K^* = Total backwater coefficient as determined from U. S. Bureau of Public Roads data.

α = Kinetic energy coefficient.

A_n = Gross water area in the bridge measured below normal water surface elevation (Sq. Ft).

Vn = Average velocity of flow through the bridge (fps) computed as:

$$Vn = \frac{Q}{An}$$

where:

Q = Assumed flow through the bridge.

A_1 = Gross water area downstream from the bridge at a location where the water is flowing at the normal elevation (Sq. Ft).

A_2 = Gross water area upstream of the bridge including that area produced by the backwater (Sq. Ft).

g = Gravitational constant.

The water surface elevation immediately upstream from a bridge was calculated by adding the computed backwater to the normal water surface elevation. Using the computed water surface elevation upstream from the bridge as the known elevation, and shifting the Kn curve to a point mid-way between the bridge and the upstream end of the reach, the water surface elevation at the upper end of the reach was then computed as previously described for a reach with no constriction. It should be noted that the Kn curve was not shifted horizontally, rather it was shifted along the line representing the average low-flow water surface slope in the reach.

If a channel constriction was provided with more than one bridge opening, the backwater calculations for the constriction were accomplished by first assuming a flow through each bridge. Backwater computations were then made for each bridge. If the computed backwater elevations agreed within ± 0.20 foot, the backwater computed for the major bridge was assumed correct.

When the normal water surface elevation plus the computed backwater from a bridge resulted in a water surface elevation that would cause flow over a road or railroad embankment, a flow through the bridge and a corresponding flow over the embankment were assumed. The backwater caused by the bridge was then calculated. The flow over the embankment was then calculated according to formulae determined by Yarnell (13) as follows:

For road embankments:

$$Q = CLH^{1.60} \text{ - } (8)$$

where:

Q = Flow over the embankment (cfs).

C = A constant whose value varies with the degree of submergence. For no submergence, C = 2.66. Values for various degrees of submergence are given by Yarnell.

L = Length of embankment over which flow is assumed (Ft).

$$H = D + \frac{V^2}{2g}$$

where:

D = Depth of flow over the embankment, measured from the crown of the road (Ft).

V = Average velocity of water approaching the embankment (fps).

g = Gravitational constant.

For railroad embankments:

$$Q = CLH^{1.47} \text{ - - - - - - - - - - - - - - - - } (9)$$

where:

Q = Flow over the embankment (cfs).

C = A constant whose value varies with the degree of submergence, $C = 3.27$. Values for various degrees of submergences are given by Yarnell.

L = Length of embankment over which flow is assumed (Ft).

$$H = D + \frac{V^2}{2g}$$

where:

D = Depth of flow over the embankment, measured from the top of rail (Ft).

V = Average velocity of water approaching the embankment (fps).

g = Gravitational constant.

If the calculated and assumed flow over the embankment agreed within $\pm$ 5 per cent, the backwater elevation calculated for the bridge was assumed correct.

Flow through culverts under constricting embankments was considered only for culverts larger than 48 inches in size. Trial calculations considering the smaller culverts indicated that the flow through the culverts was such a small fraction of the total flow that backwater relief was negligible.

The method of computing backwater at constrictions with large culverts was to divide the total flow between the bridge and the culvert.

The backwater elevation was computed for the flow through the bridge. The discharge through the culvert was then determined by means of head-discharge tables given by Chow (10). If the calculated flow was within $\pm$ 5 per cent of the assumed flow, the calculated backwater from the bridge was considered correct.

SAMPLE CALCULATIONS

Water surface profile calculations will be demonstrated for a reach of stream which starts 100' downstream of South Riverside Drive and ends 795' upstream of South Riverside Drive in order to illustrate the methods of calculation used for this study. A flow of 6,200 cfs in Squaw Creek will be used in this example. The water surface elevation at the lower end of the reach is fixed at 66.66 by calculations made previously. The length of the reach is 895'.

It is first necessary to plot a cross section for each end of the selected reach. Figures 13 and 14 are reduced scale reproductions of the actual cross sections used for this study.

The cross sections provide the topographic information required for calculation of the conveyance at various elevations using Equation 3. Tables 2 and 3 show the data obtained and calculations made for the two end sections.

Using Equation 4, the mean conveyance of the reach was calculated as shown in Table 4, and a curve of Km vs. water surface elevation constructed as shown in Figure 12.

The normal water surface profile, neglecting the constriction, can now be determined. Assume the water surface elevation at the mid-point of the reach as 67.00. From Figure 12, Km is found to be 220,000. Using Equation 5, the slope is calculated to be:

$$S = \left(\frac{6,200}{220,000}\right)^2 = 0.000766$$

Table 2. Conveyance of section 100' downstream from South Riverside Drive

Elevation	Location and Description	n	A sq. ft.	P ft	R ft	$R^{2/3}$	Conveyance $(K = \frac{1.486}{n} AR^{2/3})$
60	Channel	0.04	480	115	4.17	2.59	45,200
62	Channel	0.04	700	130	5.38	3.07	80,000
64	Channel	0.04	955	150	6.37	3.44	122,000
66	Channel	0.04	1,240	160	7.75	3.92	181,000
	North O'Bank - Scrub	0.11	40	80	0.50	0.63	300
	TOTAL						181,300
68	South O'Bank - Cultivated	0.06	6,235	3,100	2.00	1.59	245,000
	South O'Bank-Timber & Scrub	0.12	1,350	510	2.65	1.92	32,100
	Channel	0.04	1,720	160	10.75	4.87	311,500
	North O'Bank - Scrub	0.11	490	250	1.96	1.52	10,400
	TOTAL						599,000
70	South O'Bank - Cultivated	0.06	12,405	3,150	3.94	2.50	747,000
	South O'Bank-Timber & Scrub	0.12	2,350	510	4.61	2.77	80,900
	Channel	0.04	2,200	160	13.75	5.74	468,000
	North O'Bank - Scrub	0.11	1,070	350	3.06	2.11	30,400
	TOTAL						1,326,300

Table 2. (Continued)

Elevation	Location and Description	n	A sq. ft.	P ft.	R ft.	$R^{2/3}$	Conveyance $(K = \frac{1.486}{n} AR^{2/3})$
72	South O'Bank - Cultivated	0.06	18,685	3,210	5.81	3.25	1,495,000
	South O'Bank-Timber & Scrub	0.12	3,350	510	6.57	3.51	146,000
	Channel	0.04	2,680	160	16.75	6.55	652,000
	North O'Bank - Scrub	0.11	1,825	450	4.05	2.54	62,500
	TOTAL						2,355,500
74	South O'Bank - Cultivated	0.06	25,075	3,270	7.66	3.89	2,420,000
	South O'Bank-Timber & Scrub	0.12	4,350	510	8.53	4.18	225,000
	Channel	0.04	2,160	160	19.75	7.31	859,000
	North O'Bank - Scrub	0.11	2,740	550	4.98	2.92	108,000
	TOTAL						3,612,000

Table 3. Conveyance of section 795 feet upstream from South Riverside Drive

Elevation (City datum)	Location and Description	n	A sq ft	Wetted Perimeter P (ft)	$R = \frac{A}{P}$ ft	$R^{2/3}$	Conveyance $(K = \frac{1.486}{n} AR^{2/3})$
60	Channel	0.04	255	80	3.19	2.17	20,600
62	Channel	0.04	415	115	3.61	2.35	36,300
64	West O'Bank - Timber & Grass	0.10	130	140	0.93	0.95	1,800
	Channel	0.04	600	118	5.07	2.95	65,900
	TOTAL						67,700
66	West O'Bank - Timber & Grass	0.10	70	140	0.50	0.63	700
	West O'Bank - Timber & Grass	0.10	50	100	0.50	0.63	500
	West O'Bank - Timber & Grass	0.10	204	125	1.63	1.38	4,200
	West O'Bank - Timber & Grass	0.10	365	140	2.61	1.90	10,400
	Channel	0.40	790	120	6.58	3.51	101,500
	TOTAL						117,300
68	West O'Bank - Cultivated	0.06	1,765	1,270	1.39	1.24	54,200
	West O'Bank - Timber & Grass	0.10	1,575	580	2.71	1.94	45,500
	Channel	0.04	985	125	7.88	3.96	145,000
	TOTAL						244,700

Table 3. (Continued)

Elevation (City datum)	Location and Description	n	A sq ft	Wetted Perimeter P (ft)	$R = \frac{A}{P}$ ft	$R^{2/3}$	Conveyance $(K = \frac{1.486}{n} AR^{2/3})$
70	West O'Bank - Cultivated	0.06	4,350	1,350	3.22	2.18	235,000
	West O'Bank - Timber & Grass	0.10	2,675	580	4.61	2.77	110,500
	Channel	0.04	1,185	128	9.26	4.42	195,000
	TOTAL						540,500
72	West O'Bank - Cultivated	0.06	7,040	1,400	5.01	2.93	505,000
	West O'Bank - Timber & Grass	0.10	3,775	580	6.50	3.48	196,000
	Channel	0.40	1,385	128	10.80	4.69	252,000
	East O'Bank - Scrub	0.11	150	160	0.93	0.95	1,600
	TOTAL						954,600
74	West O'Bank - Cultivated	0.06	9,800	1,425	6.87	3.61	875,000
	West O'Bank - Timber & Grass	0.10	4,875	580	8.40	4.13	301,000
	Channel	0.04	1,585	128	12.40	5.36	316,000
	East O'Bank - Scrub	0.11	350	260	1.35	1.22	5,700
	TOTAL						1,497,700

Table 4. Mean conveyance of reach between 100' downstream from South Riverside Drive and 795' upstream of South Riverside Drive

Elevation	Conveyance at Lower end of Reach Kd	Conveyance at Upper end of Reach Ku	Mean Conveyance $\sqrt{KuKd}$
60	45,200	20,600	30,600
62	80,000	36,300	53,000
64	122,000	67,700	90,800
66	181,400	117,300	146,000
68	599,000	244,700	381,000
70	1,326,300	540,500	855,000
72	2,355,500	954,600	1,480,000
74	3,612,000	1,497,700	2,320,000

The rise in water surface from the lower end of the reach to the mid-point is then given by Equation 6:

$$R = 0.000766 \ \frac{895}{2} \ = 0.34'$$

The calculated water surface elevation at the mid-point is then equal to the elevation at the lower end of the reach plus the calculated rise:

Elevation at mid-point = 66.66 + 0.34 = 67.00

which is the assumed elevation, therefore the computed slope is assumed correct.

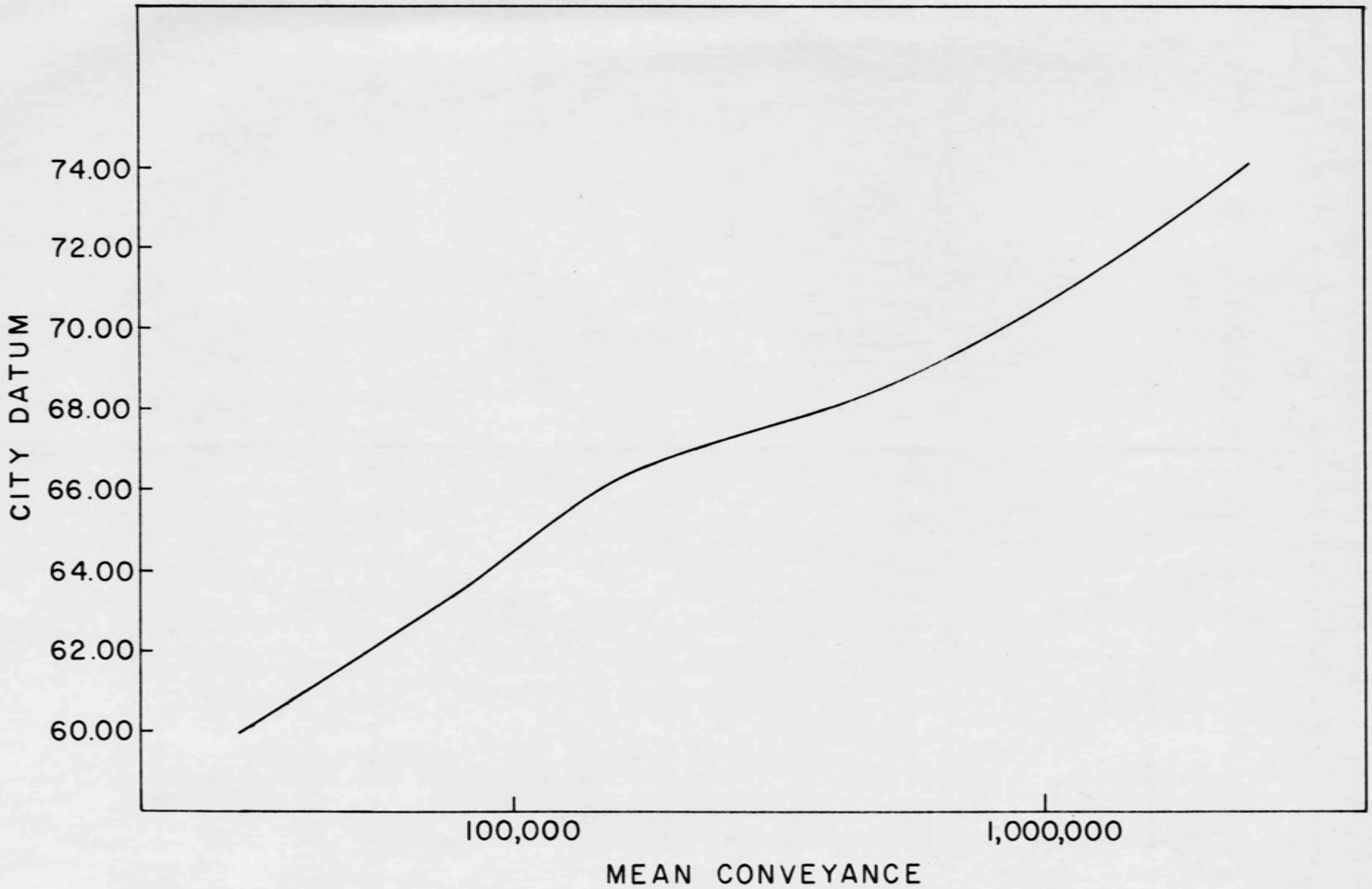


Figure 12. Water surface elevation vs. mean conveyance

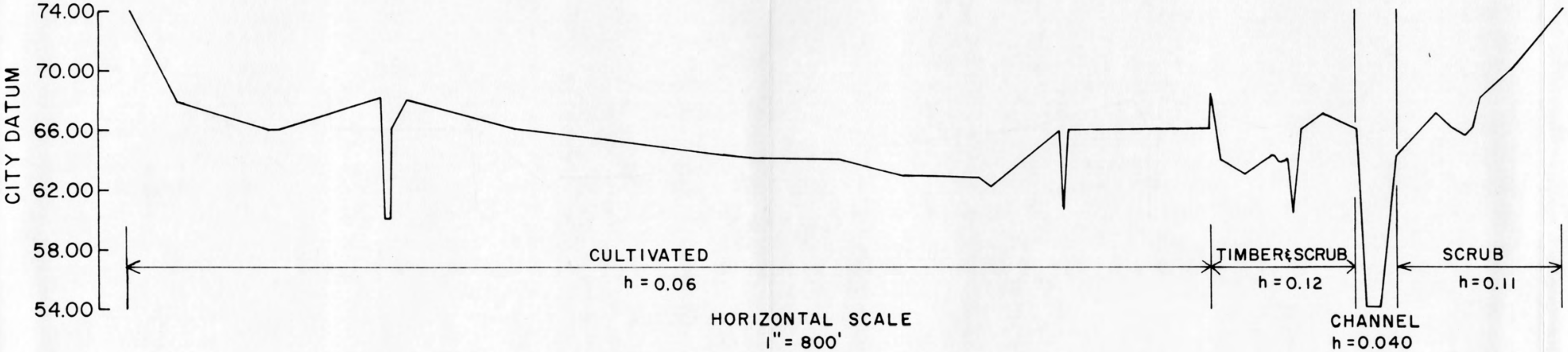


Figure 13. Cross section of Squaw Creek and flood plain 100' downstream from South Riverside Drive

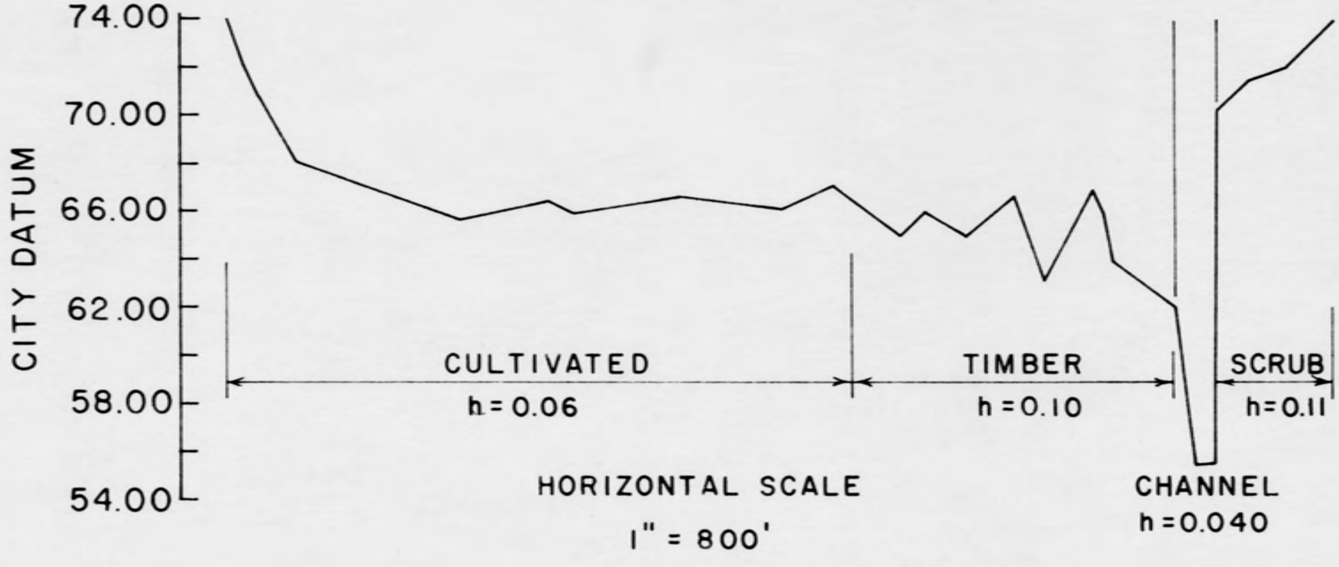


Figure 14. Cross section of Squaw Creek and flood plain 795' upstream from South Riverside Drive

The total rise in water surface elevation in the reach can be calculated as:

$$R_{TOT} = 0.000766\ (895) = 0.68'$$

The water surface elevation at the upper end of the reach is then found to be:

$$\text{Elevation at upper end of reach} = 66.66 + 0.68 = 67.34$$

If the reach had not contained a constriction, the water surface profile would have been plotted, and calculations made for the next reach upstream. However, since the South Riverside Drive bridge is included in the reach, the backwater resulting from the constriction must be calculated.

The normal water surface elevation at South Riverside Drive bridge can be calculated by interpolation. Since the total rise in water surface elevation in 895 feet is 0.68 foot the rise in 100 feet is:

$$\text{Rise} = 0.68\ \left(\frac{100}{895}\right) = 0.08\ \text{Ft.}$$

The normal water surface elevation at South Riverside Drive bridge is then found to be:

$$66.66 + 0.08 = 66.74$$

With the normal water surface elevation computed, the backwater can be calculated according to Equation 7.

However, since a flow of 6,200 cfs causes overtopping of South

Riverside Drive, the flow must be assumed to be divided. For this study, 5800 cfs was assumed as passing through the bridge and 400 cfs over the road embankment.

The backwater caused by the constriction was computed by Equation 7 and found to be 0.64 foot and the water surface elevation immediately upstream of the bridge:

$$66.74 + 0.64 = 67.38$$

With this water surface elevation it was determined that 880 feet of road would be overtopped with an average depth of 0.60 foot of water, with no submergence. By Equation 8, the flow over the road can be calculated to be:

$$Q = 2.66\ (880)\ (0.60)^{1.60} = 411 \text{ cfs}$$

Since this agrees, within the desired limits, with the assumed flow of 400 cfs, the calculated backwater from the bridge is assumed correct.

The water surface elevation at the upper end of the reach can now be calculated by assuming the bridge to be the lower end of the reach, and the known water surface elevation of 67.38 as the fixed elevation. The distance to the upper end of the reach is 795 feet from the bridge.

Assume the water surface elevation at the mid-point between the bridge and the upper end of the reach to be 67.60. The conveyance at that elevation can be ob tained by shifting the Km vs. water surface elevation curve for the original reach 50 feet upstream at the low water slope. In effect this can be done by reading the Km value at an elevation 0.10 foot lower than the elevation desired, as the shifting of the curve 50 feet in the reach is essentially the same as raising the curve 0.10 foot. In order to obtain the conveyance at elevation 67.60, Figure 12 is entered at elevation 67.50, and Km is found to be 280,000.

From Equation 5, the average water surface slope in the reach is:

$$S = {\frac{6,200}{280,000}}^{2} = 0.000495$$

From Equation 6, the rise in water surface elevation in one half the reach is:

$$R = 0.000495 \; \frac{795}{2} = 0.196$$

The calculated water surface elevation at the mid-point becomes:

$$67.38 + 0.20 = 67.58$$

Since this is within desired limits, the slope is assumed correct and the total rise of the water surface in the reach becomes:

$$R_{TOT} = 0.000495\ (795) = 0.39$$

and the elevation at the upper end of the reach is:

$$67.38 + 0.39 = 67.77$$

With this elevation known, the water surface profile calculations may be continued upstream.

DISCUSSION

The choice of variables in all calculations for this study was predicted upon summer conditions, i.e., growing crops in cultivated areas, trees and shrubs fully leafed, etc. This was considered to be a logical approach since the majority of past flooding has occurred during the summer months.

Although the choice of variables such as the roughness coefficient in Manning's equation may be chosen artificially high in order to maximize water surface elevations, this was not done in this study. The variables were chosen based on the best judgement of the author after study of several sources.

Allowances were made in backwater calculations for a slight accumulation of debris on bridge piers. It is possible for large quantities of debris to accumulate on the piers which would cause backwater and possibly flooding in excess of normal. However, the probability of this occurrence cannot be estimated. Design engineers using this data should make allowance for this occurrence by adding freeboard above that indicated by the flood profile curves.

Although the basic data used in this study are believed to be the best available and calculations were made and checked, it should be realized that many estimates requiring considerable engineering judgement are involved, and the results are subject to the limitations of basic data and the brief amount of time available for completion of the study. The results are presented as a preliminary estimate of the results of flood events that may occur in the future.

The results of this study are valid only for the present conditions in the flood plain. Any future construction that tends to encroach and restrict the flow of water either in the channel or in the flood plain will tend to alter the results of this study and increases in flood stages will occur.

The location of the U. S. Geological Survey gage was taken as the lower end of the first reach considered in this study since the elevation at that point could readily be fixed by reading the gage rating curve or the extension of the curve when required. Backwater effects of the U. S. Highway #30 By-pass Bridge to be constructed approximately one quarter mile downstream from the gaging station site were not considered in fixing the water surface elevations at the gaging station. Information from Iowa Natural Resources Council (14) indicates that a maximum of approximately 1.00 Ft. backwater could be expected from the new bridge. Trial calculations showed that a 1.00 foot elevation difference at the gaging station caused only a change of approximately 0.10 foot at South Duff Avenue on Squaw Creek. It was then concluded that backwater effects of the U. S. Highway #30 By-pass Bridge could be neglected for this study.

RESULTS AND CONCLUSIONS

The results of bridge backwater calculations are shown in Table 5. An estimate of the validity of these calculations was made by comparing the computed difference in water surface elevations between various bridges with the difference in elevations observed during past floods by Dr. Harris F. Seidel. This comparison was made using the difference between calculated elevations for flood flows of less than 10,000 cfs with the differences in elevation computed from the data in Table 1. The results are tabulated below:

Difference in water surface elevation between bridges as observed and computed for various floods

Between Bridges Located at:	Difference in Water Surface Elevation	
	Observed by Dr. Harris F. Seidel (Ft)	Computed (Ft)
13th Street and 6th Street	3.1 - 4.0	3.0 - 4.0
6th Street and Lincoln Way	2.3	2.1 - 3.1
Lincoln Way and South Riverside Drive	1.9 - 2.9	2.2 - 3.5
South Riverside Drive and South Duff Avenue	7.2 - 7.9	6.0 - 7.1

The purpose of this comparison was to determine if calculated differences in elevation compared reasonably with observed differences. It was concluded that a good correlation existed between calculated and observed elevation differentials.

The results of water surface profile calculations are shown in Figures 15 through 22. Flood profiles for three floods through the entire reach are shown in Figure 15. The remaining figures show computed water profiles by reaches. The figures indicate the water surface elevations to which flooding can reasonably be expected to occur as well as the frequency of occurrence. It should be noted that the distances indicated on these figures are stream centerline distances, which are generally greater than the corresponding valley distances.

Data from Figures 15 through 22 may be used to estimate the depth and frequency of flooding to be expected at any point in the Squaw Creek flood plain within the City of Ames.

An example of this use is shown in Figure 25. This figure shows the relationship between the average ground elevation and the water surface elevation and percent chance of annual occurrence for a point 100 yards south of Lincoln Way. From this figure, a design engineer may easily determine the water surface elevation for which protection must be provided in planning a structure at this location. For example, a design for which protection against a water surface elevation of 2 per cent chance of annual occurrence is desired, should provide for an appropriate freeboard above elevation 69.6. This protection could either be provided by establishing a minimum floor elevation above 69.6, or by construction of a dike around the structure. Caution should be exercised, however, as either a large dike or a large area of fill in this area may restrict the flood plain to such an extent that a constricting effect is introduced, causing even higher backwater than provided for in these calculations.

Table 5. Results of bridge backwater computations

Flow (cfs)	4,700	5,400	6,200	7,600	10,000	18,600	37,300
			SOUTH DUFF AVENUE				
Normal water surface	58.76	59.32	59.64	60.15	61.11	62.27	65.36
h*	0.45	0.32	.88	.79	1.46	2.11	1.78
Upstream water surface	59.21	59.64	60.52	60.94	62.57	64.38	67.14
			RAILROAD BRIDGE				
Normal water surface	63.25	63.63	64.09	64.52	65.72	66.12	68.88
h*	0.83	1.58	1.84	1.69	1.45	2.55	1.46
Upstream water surface	64.08	65.21	65.93	66.21	67.17	68.67	70.34
			SOUTH RIVERSIDE DRIVE				
Normal water surface	65.15	65.97	66.74	66.99	67.66	69.20	71.25
h*	0.55	0.55	0.64	1.01	0.94	0.52	0.14
Upstream water surface	65.70	66.52	67.38	68.00	68.60	69,72	71.39
			LINCOLN WAY BRIDGE				
Normal water surface	67.55	68.16	68.77	69.38	70.10	71.28	73.28
h*	9.42	0.15	0.55	0.74	0.29	0.77	1.71
Upstream water surface	67.97	68.31	69.32	70.12	70.39	72.05	74.99
			6th STREET BRIDGE				
Normal water surface	70.25	70.74	71.35	71.83	72.31	73.99	76.54
h*	0.03	0.11	0.85	1.19	0.22	1.30	3.29
Upstream water surface	70.28	70.85	72.20	73.02	72.53	75.29	79.83

Table 5. (Continued)

			13th STREET BRIDGE				
Normal water surface	73.85	74.25	74.71	75.26	75.85	77.92	81.78
h*	0.38	0.32	0.52	0.74	0.20	1.23	2.70
Upstream water surface	74.23	74.57	75.23	76.00	76.05	79.15	84.48
			STANGE ROAD BRIDGE				
Normal water surface	76.53	76.55	77.19	78.03	79.10	81.25	86.68
h*	0.23	0.27	0.34	0.51	0.33	1.00	0.22
Upstream water surface	76.76	76.82	77.53	78.54	79.43	82.25	86.90

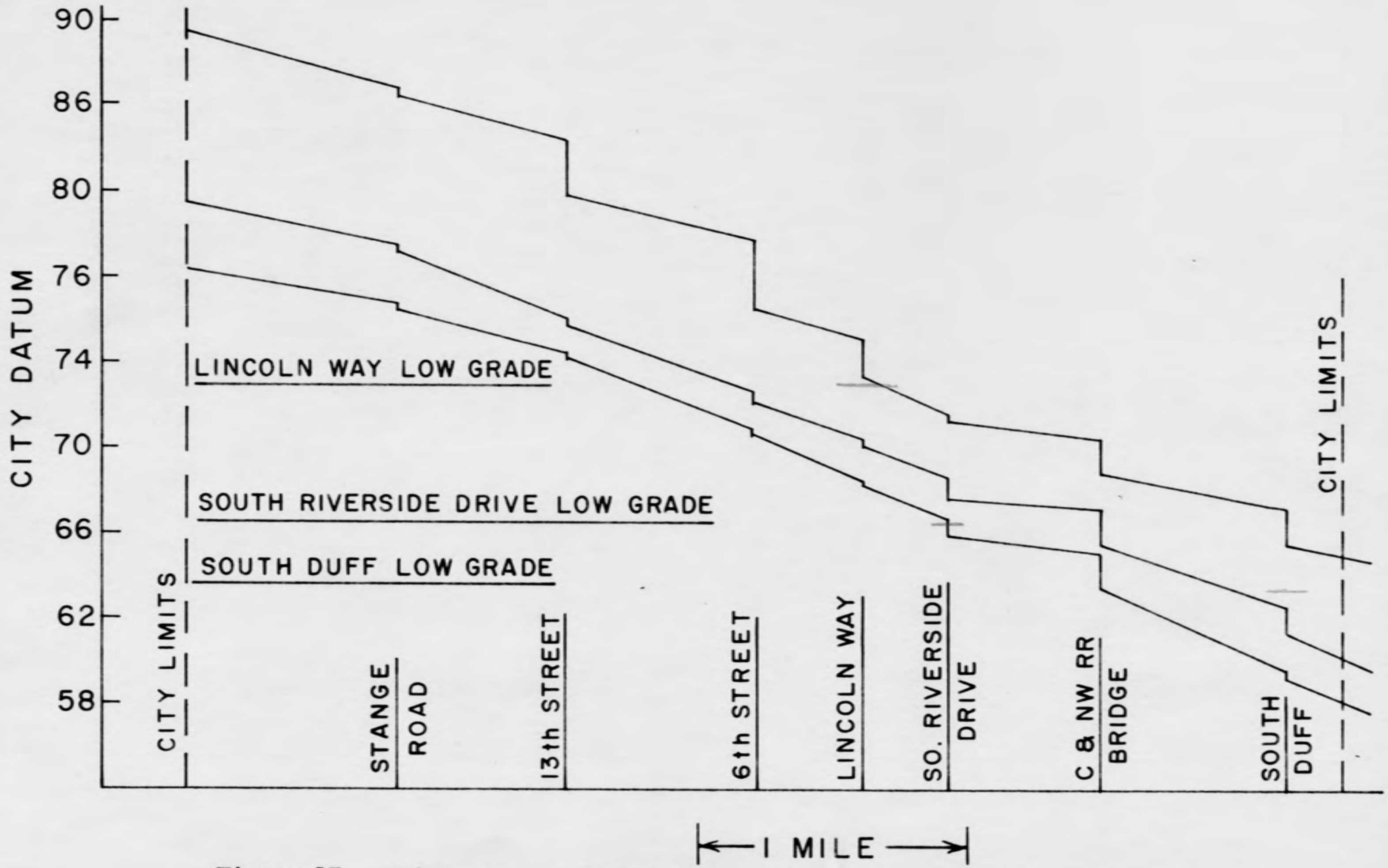


Figure 15. Water surface profiles for floods of various frequency

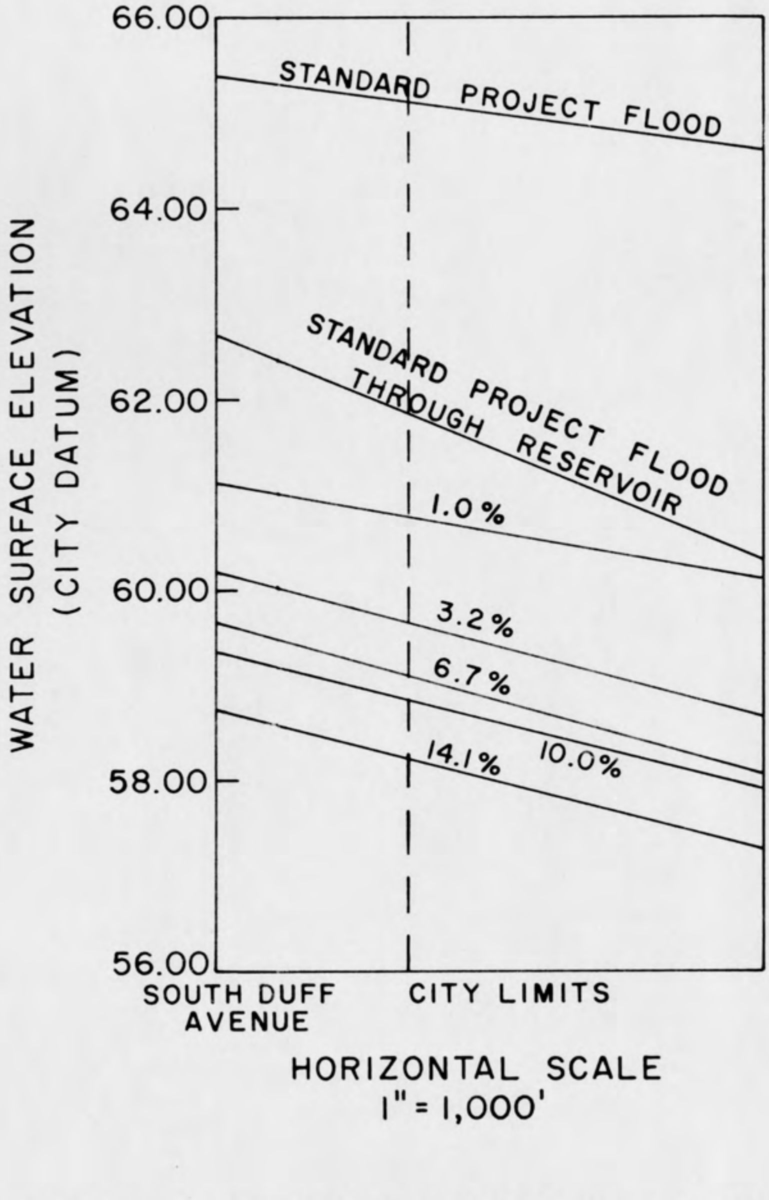


Figure 16. Water surface profiles of various per cent chance of annual occurrence: city limits to South Duff Avenue

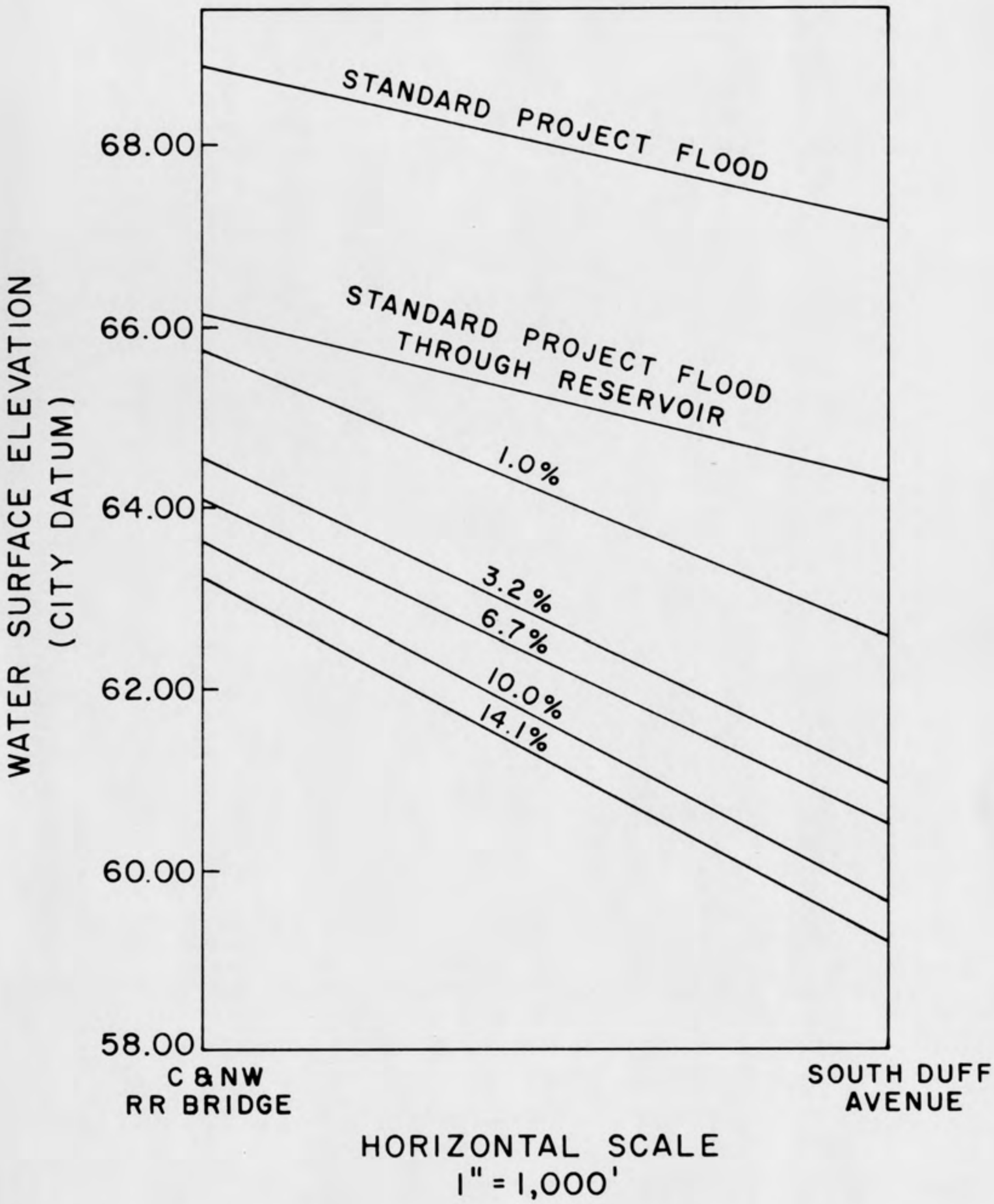


Figure 17. Water surface profiles of various per cent chance of annual occurrence: South Duff Avenue to C & NW RR Bridge

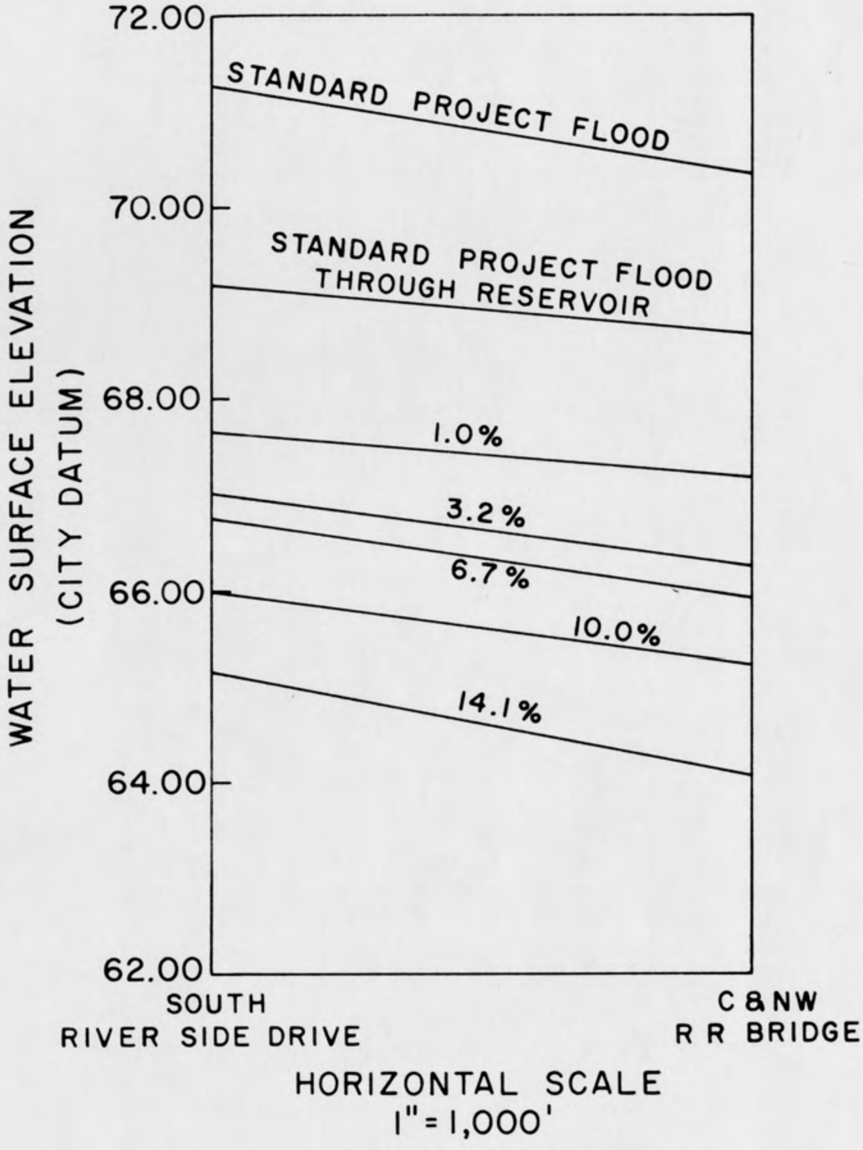


Figure 18. Water surface profiles of various per cent chance of annual occurrence: C & NW RR BRIDGE to South Riverside Drive

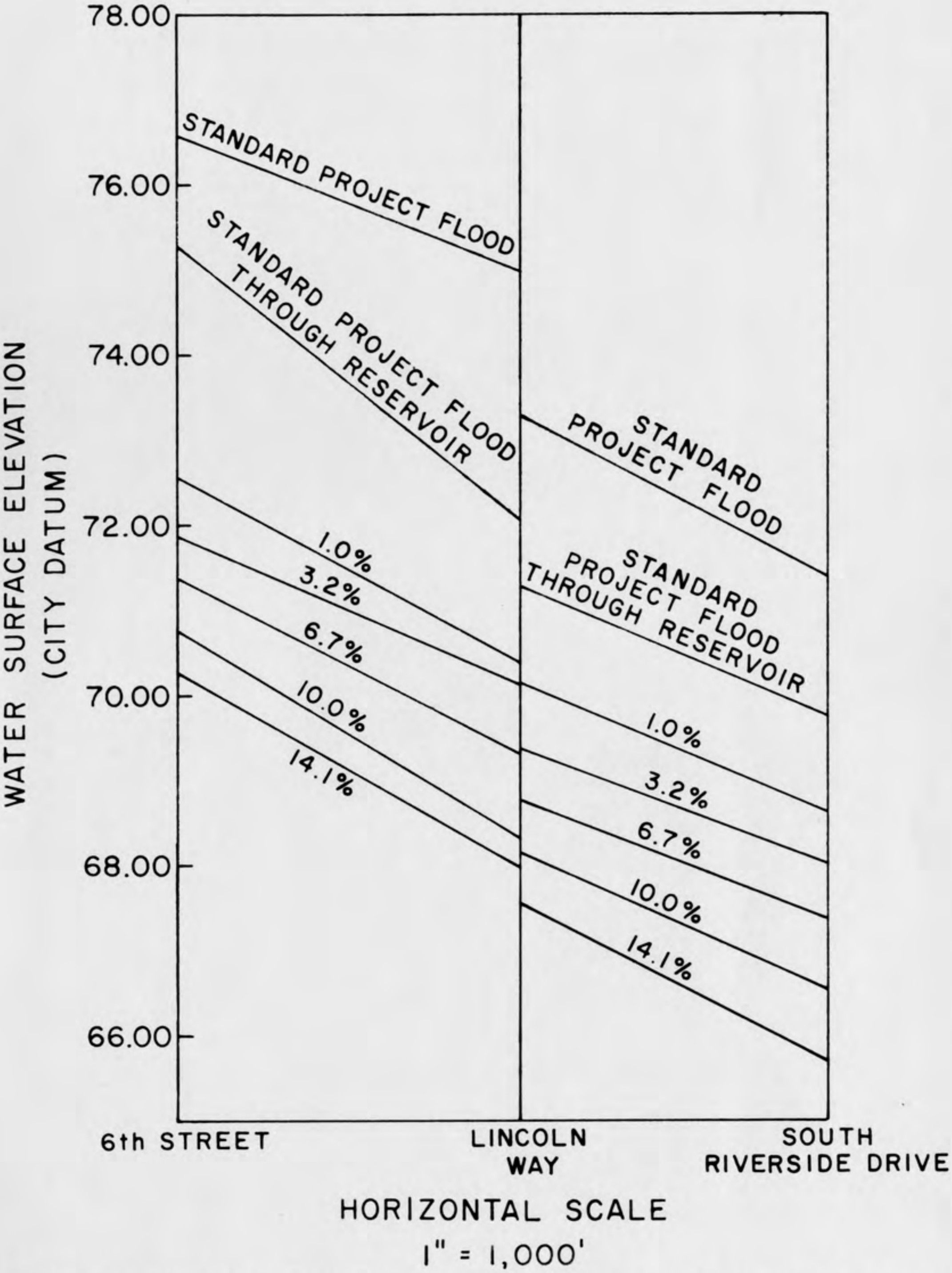


Figure 19. Water surface profiles of various per cent chance of annual occurrence: South Riverside Drive to 6th Street

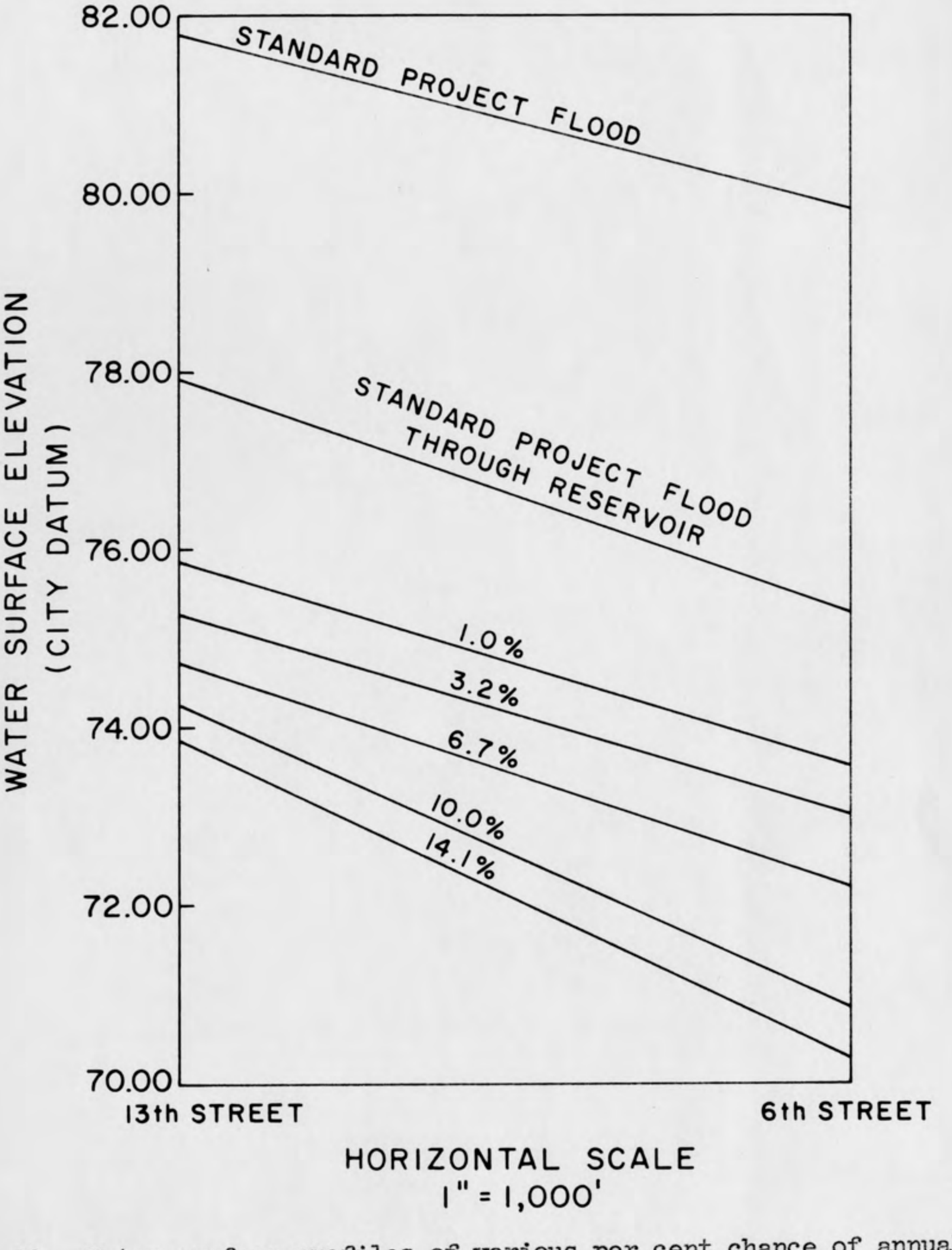


Figure 20. Water surface profiles of various per cent chance of annual occurrence: 6th Street to 13th Street

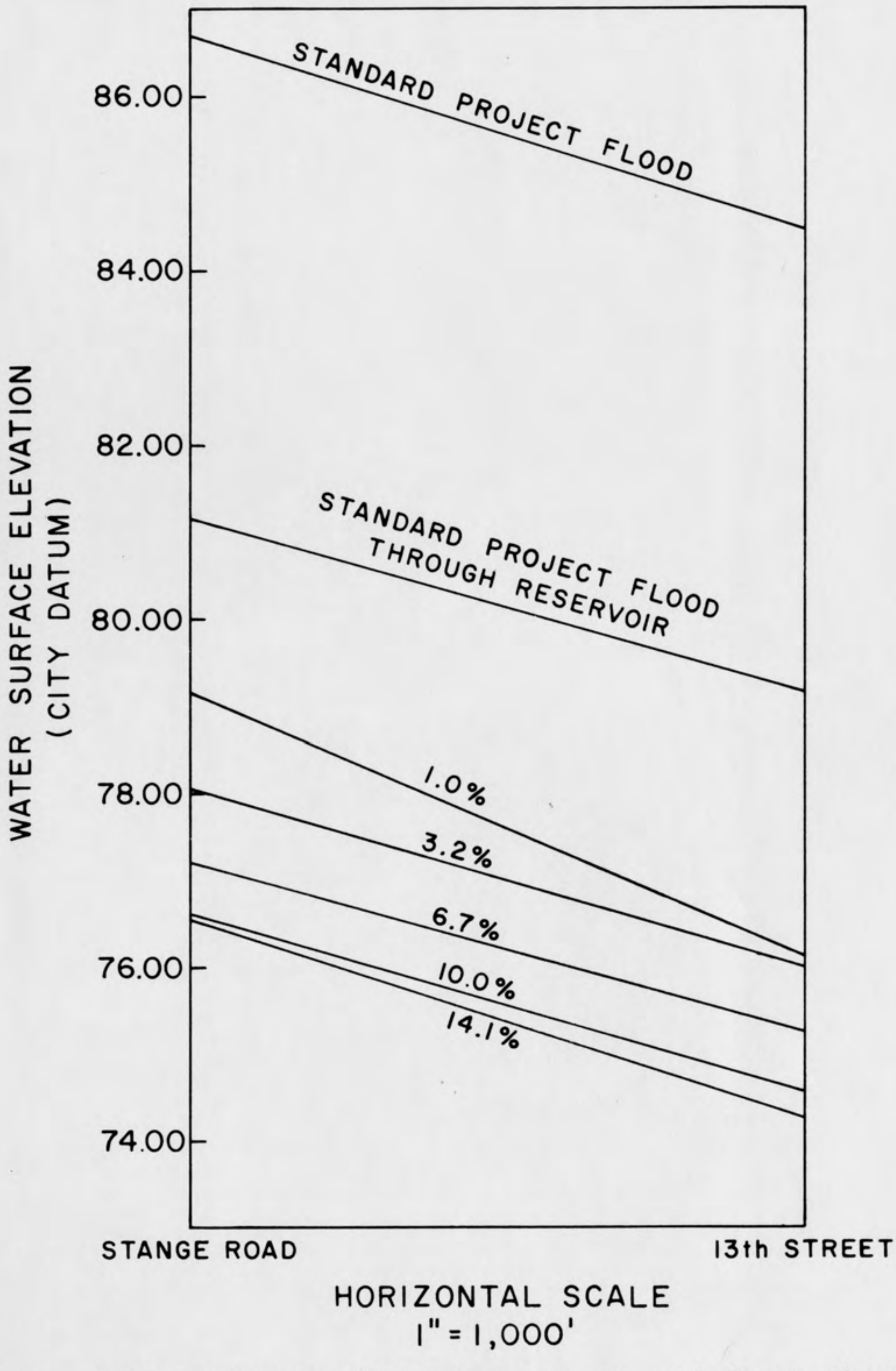


Figure 21. Water surface profiles of various per cent chance of annual occurrence: 13th Street to Stange Road

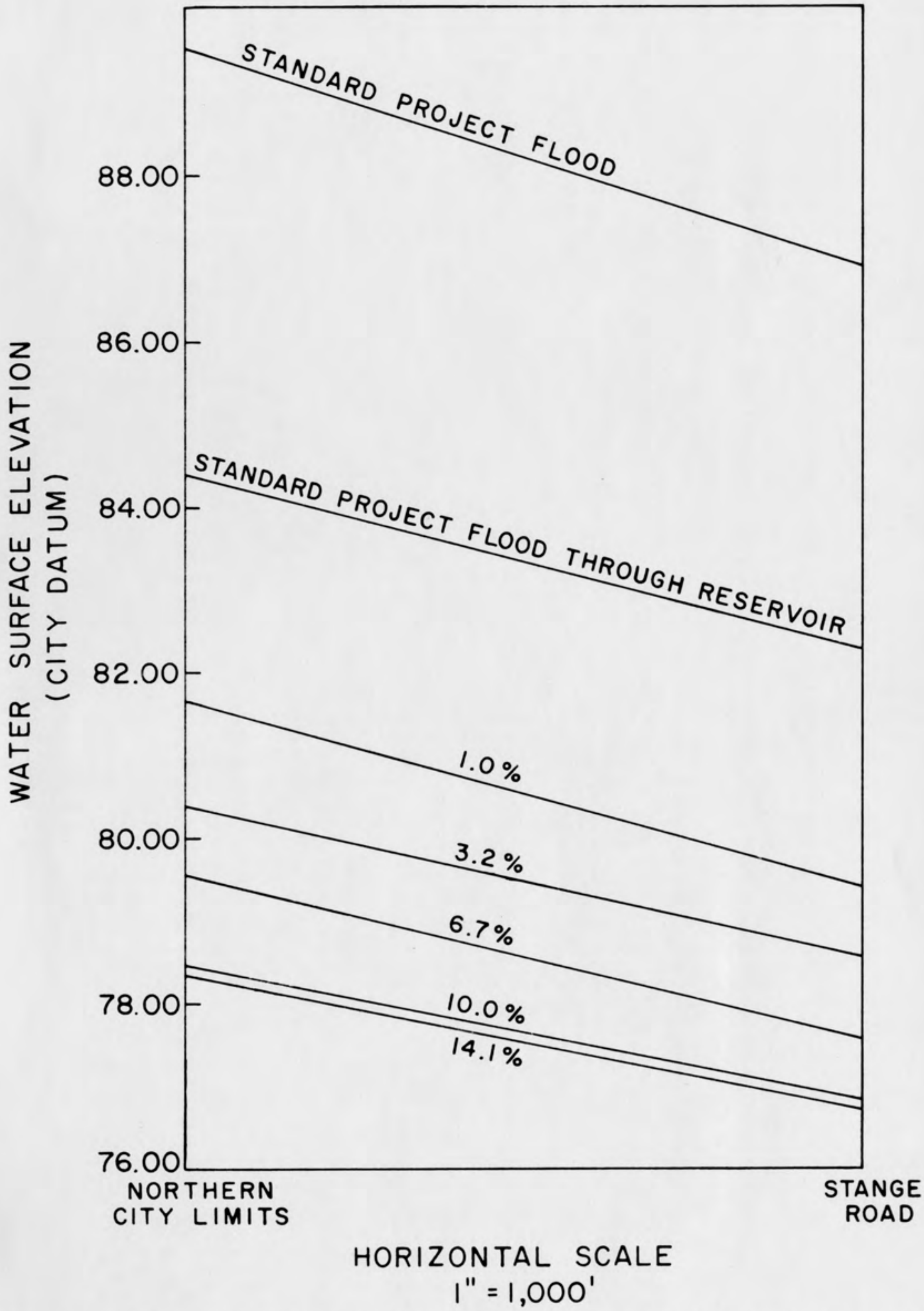


Figure 22. Water surface profiles of various per cent chance of annual occurrence: Stange Road to Northern city limits

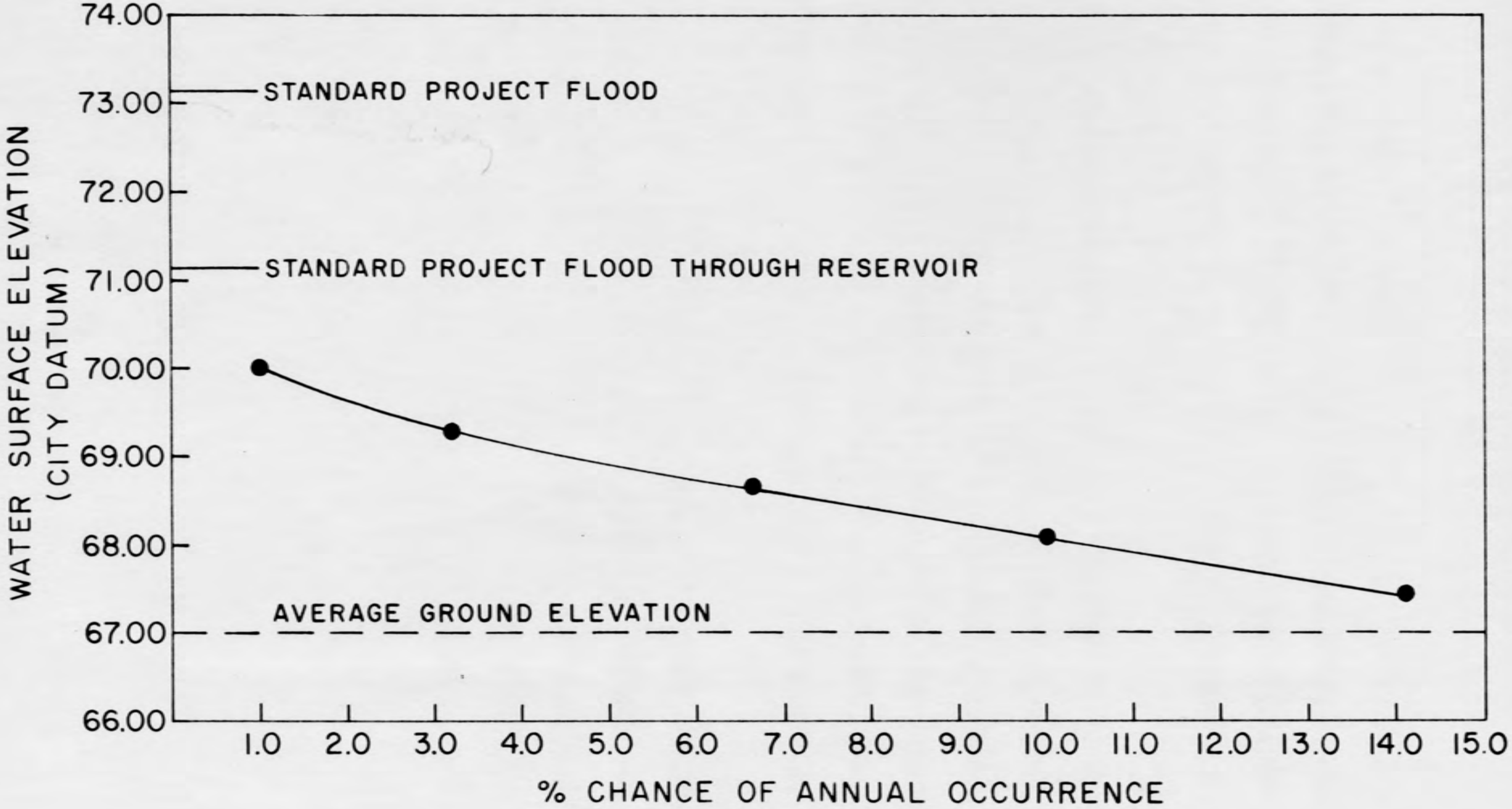


Figure 23. Flood depth-frequency at a point 100 yards south of Lincoln Way

RECOMMENDATIONS

It is recommended that future construction in the flood plain of Squaw Creek within the City of Ames be restricted by appropriate legal methods. The restrictions should either prohibit construction of facilities susceptible to serious damages by flood waters, or provide for minimum floor elevations for proposed structures.

It is also recommended that the results of this study be used as a guide for establishing the restrictions to be placed upon future construction in the flood plain. All such restrictions should include a reasonable freeboard above the water surface elevations determined by this study. A minimum freeboard of 2 feet is recommended unless proposed structures will be of such construction and used for such purposes that short periods of inundation would result in only minor damages.

If restriction or regulation of construction in the flood plain is determined undesirable by those agencies responsible for land use control, the City of Ames may still consider it advisable to adopt a policy of informing responsible individuals of the flood hazard to be expected in the flood plain at the time application for a building permit is made.

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